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Progress on Catskill Aqueduct One of the favorite expressions of the chief engineer of the Board of Water Supply of New York, "Get it done!", means a great deal, if the speed record of the Catskill works is compared with progress on the Croton system. The Croton Aqueduct Commission was authorized by the legislature of 1883, and finished a new aqueduct to the city in June, 1890. This was a good record for seven years, as the distance covered is 33¼ miles; but thereafter the commission nursed its job most assiduously. The completion of the new Croton dam, started in 1892, was not accomplished until 1907, and it was not until a suit was instituted against the commission in 1909 by the mayor of New York that the affairs of that body were closed up permanently. The Board of Water Supply was appointed in June, 1905, and it has finished the Ashokan dam and cleaned up the aqueduct, so that in a very short time it will be ready for emergency use. The main aqueduct is 92 miles long, nearly three times the length of the new Croton aqueduct, not including the New York City tunnel, which was authorized just three years ago and was holed through under the East River on Sept. 27 of the current year. Such a complete change in the speed record of the two commissions, one organized before the other was driven out of office, has a significance to the taxpayers which they doubtless fail to appreciate but is worth mentioning—this is the saving in interest charges resulting from the prosecution of work of such magnitude on such a definite-time schedule. The completeness of the Catskill work at the present rate has a financial side as interesting as its engineering aspects.

Parks versus Harbors Since the War Department Engineers are averse to the park commissions of Chicago usurping practically all of the lakefront from the mouth of the Chicago River south various plans have been proposed to effect a compromise between the Federal authorities, who are looking primarily after interstate shipping interests, and those who are desirous of parking large stretches of the waterfront. A conference is to be held in Washington in November between the local representatives and the Secretary of War. One of the compromise plans by Mr. John M. Ewen, chairman of the Harbor Commission in 1909, advocates using the shore for park development until such time as shipping may need the space for docks. The park enthusiasts and Mayor Harrison are of the opinion that lake commerce will never grow to such proportions as to need more than the 2 miles of waterfront north and south of the river. The harbor proponents are more sanguine, and Mr. Ewen states that "time only will tell," and for that reason he believes the park development should be made so that future docks may be constructed if required, the city being given the right to do so whenever it sees the need thereof. If the railroad-terminal plan proposed by Mr. John F. Wallace should ever materialize and a great station be built at Twelfth Street, docks in this vicinity would certainly be desirable, but in the meantime there is no reason why the city should not use the frontage as a playground. Already Chicago has committed itself by bond issues to that part of the 1909 Harbor Commission report relating to the replacement of the center span bridges over the river and construction of docks north of the river. It would seem proper to make further use of this valuable report and go to Washington with a plan to which there is reasonable assurance of acquiescence by the War Department.

"Well Done" The closing chapters of the Los Angeles Aqueduct work, marked by the dedication ceremonies already recorded in this journal, call forth a tribute to those who have borne the brunt in the mighty achievement. Those with experience in large public works could have predicted on the day the plan was finally adopted that those who undertook the burdens would be obliged to weather a goodly share of criticism, for somehow scandalmongers, "soreheads," politicians and incompetents will arise to cast suspicion on the most honest intentions, so long as the intentions are those of officials intrusted with the expenditure of millions of the people's money. The officials of the Los Angeles Aqueduct have been subjected to this criticism, but the citizens, having faith in the competence of the engineers in charge, ignored the charges or duly weighed their source. The toilers continued their even way, and public opinion, if at any time even wavering, returned to back them. Now that the work is practically finished and the pipe of plenty stretched across 234 miles of mountains, plain and desert, the engineering corps can look with well-merited satisfaction at its handiwork. To the leaders, William Mulholland, chief engineer, and J. B. Lippincott, assistant chief engineer, the principal credit is due. They pointed the way, set the pace and inspired the organization. Without loyalty in the staff below and perseverance in the face of hardship, however, the work could not have been put through. In full appreciation of this the leaders will not begrudge a good meed of praise to the individuals in the ranks, even to those who almost literally were the hewers of wood and the drawers of water.

San Francisco Dock Failure No fatalities resulted from the collapse of the Green Street Wharf in San Francisco on Nov. 11, as noted in the Current News Section of this issue, but the failure does not reflect credit on the California State Harbor officials. It is true that the wharf in question, as well as a number of similar tottering, teredo-eaten structures, were left as an onerous and very unwelcome heritage to the present Board of Harbor Commissioners. For weeks in the daily press the members decried some of the docks that had been allowed to stand. That was more than two years ago. Since then much new construction has been undertaken and completed by the present board. This work, as far as the Engineering Record knows, is above criticism. But it seems to be a very shortsighted policy to allow docks that have been examined two years ago and found absolutely unsafe at that time to remain standing on the most prominent portion of the waterfront. They bring the State no revenue, are a constant menace to human life, and a monument to carelessness and inaction.

The condition of the Green Street Wharf was no secret. Not only had it been condemned by a competent engineer, but it could even be observed to sway dangerously when hit by moderate sized waves. There are other docks along the Embarcadero of San Francisco whose condition is almost as bad. They should not be allowed to remain until they also fall to pieces, but should be quickly rebuilt or demolished by some other agency than their own dead weight.

Competition of Foreign Cement There has been more or less speculation since the new tariff schedule was adopted as to the effect upon the American cement market. The cement companies apparently felt that the removal of the former rate would not be a sufficient inducement to bring foreign cement here in any great quantity, though there were some who felt that a sizable business might be built up at certain coast ports. That the foreign manufacturers have designs on this market, to the extent, at least, of trying their strength, is indicated by offers made last week by a Belgian company for cement delivered in the Philadelphia territory. The offerings were at 20 cents below the prices made by the mills of the Lehigh district. Just what headway was made cannot be learned. It is unlikely, however, that the foreign companies will make any impression in the industry outside the coast ports, for rail rates would soon destroy their advantage. It is quite likely, too, that factors other than American market conditions will determine the extent of the foreign invasion. The most important of these will undoubtedly be the condition of the European market. When demand there is brisk and prices good, there will be no temptation to dispose of wares on this side of the water. As a dumping point, however, for glutted home markets our coast will probably look attractive.

Great Britain's Railway Policy The appointment of a Royal Commission in Great Britain to inquire into the relations of railways and the State is given additional importance by the agitation for nationalization of these properties. The commission was appointed specifically for the purpose of inquiring into the relationship between the companies and the State concerning matters other than safety of operation and conditions of employment and to report what changes, if any, are desirable in that relationship. This, of course, might mean the study of closer regulation, but the prevailing agitation points to at least a consideration of the subject of government ownership. Two previous similar appointed bodies, which reported respectively in 1865 and 1872, touched on the subject of ownership, but did not favor it when their reports were made. Thus Great Britain, like this country, is taking up its railway problem from a new standpoint. In that country the inquiry is really a study of existing relations as the basis of changes. In this country the new attitude of the government means valuation of the properties, and already extremists want to promote agitation for government ownership. In both countries the issue of government ownership naturally raises radical views on both sides, but the other large question involved should not be lost to sight. That question is the stirring necessity of providing funds for railway development. Theoretically, government-owned enterprises need not be profitable in themselves, because deficits are absorbed in the general taxes and are therefore the burden of all the taxpaying people. If they are not profitable that fact does not affect the ability of the government, so long as it is in good credit, to borrow capital for their requirements. Individual companies, however, are forced to discontinue betterments if their earnings are not adequate, although there is always the alternative of receivership, in which case new money is available from receivers' certificates while the old capital suffers. The problem of Great Britain is much more serious than that of this country because of the greater capital costs with which English railways are loaded. Rates also are much higher there than here, as they must be to carry the heavier capitalization. The English railways have a longer history than those of this country and far greater charges for obsolescence are undoubtedly carried in their capital accounts. Therefore, if the attitude of the English authorities is such as to assure protection of the outstanding securities, the authorities of this country can deal much more rigidly with values and capital and still assure protection to security-holders. It is a commercial problem, not a political one. The obligations of both countries have depreciated and neither one could finance the purchase of the railways offhand. The safe alternative is to let them be prosperous under regulation.

Advantages of the Rolling Dam

Observers of the trend of design in hydroelectric stations need not be told that the last few years have witnessed unprecedented progress in American practice. The mere mention of the introduction of the single-runner wheel for low-head developments, the transfer of the thrust-bearing from the lower or intermediate position on the shaft to the very top, and the application of a new type of thrust-bearing without oil pumps affords ample evidence of the truth of this statement without going into less important details of equipment. Despite this rapid progress, however, a feature that has become popular abroad does not seem to have appealed to American hydraulic engineers, namely, the rolling dam. Now, however, the attitude is apparently changing. The Reclamation Service has installed one on the Boise project, described in the Engineering Record of Aug. 2, 1913, page 125. A contract has been let for the erection of another on the Grand Valley project in Colorado, while the Washington Water Power Company has made the greatest step by adopting rolling dams for the Long Lake development, near Spokane. Three rollers in the latter case are each 65 ft. long, with circular section 13 ft. in diameter, and one provided with sheath extensions to raise the water level 19 ft.

The alternative for rollers are Tainter or sliding gates. It is difficult to say exactly where the economic limit lies for the Tainter type, but it may be said with certainty that gates 30 ft. wide and 12 ft. deep are not far from the economic limit. For sliding gates of the Stoney type, those used in the Panama spillways, which are 46 ft. wide and 19 ft. high, are about as large as gates of that type can be built with economy. Furthermore, the Stoney gates, on account of their submerged rollers, with their bushed journals and roller guides, offer too many chances for trouble. Sliding gates without rollers—friction gates—are in many cases prohibited because of slowness in operation and the excessive cost of operating machinery.

The gates used at Keokuk probably indicate the maximum dimensions of the friction type. They are 30 ft. long and 11 ft. high, weighing about 30,000 lb. each. With sliding gates seldom operated, rust increases the friction so that a coefficient of one must be provided for. For the Keokuk gates this means that the operating machinery might have to lift 150,000 lb.

The Tainter gate, while presenting fewer operating difficulties, is limited by its form, at least as built to date, to comparatively small structures.

The rolling dam, on the other hand, can be built for greater heights and lengths than the types mentioned. Dams as long as 100 ft. and as high as 19 ft. are in operation, while combinations of two rollers, one above the other, are planned. The advantages of greater height and length are self-evident. The greater height permits a larger discharge capacity per unit length of dam, while the greater length decreases the number of piers. Both tend to a very material reduction of the length of spillway and, consequently, of masonry costs, while the increased cost per foot length of steelwork still leaves a large proportionate saving over other available types.

As to operation, the rolling dam also has decided advantages. The friction, so important a matter in the vertical sliding type, is practically nil (only rolling friction), while the very first movement of the structure on raising, backward and upward, tends to pull it away from ice or débris formed or collected in front of it.

It may be objected that the leakage in the rolling type is likely to be large. That objection, however, is fully disposed of by the guarantee which the manufacturers—a German firm—have given on the Long Lake rollers. They have guaranteed that the total leakage around the three will not exceed 2 sec.-ft.

As to power requirements, the Long Lake design may again be quoted. The plans call for four 8-hp motors, so arranged that each roller may be operated by either of two motors. The rollers are to be raised by electric power to their highest position, giving a clearance of 25 ft. above the spillway crest, in one hour and twenty minutes, while hand operation will require thirteen hours, with six men on each roller.

Just what significance the roller type has may be more easily grasped by comparing a roller design with the sliding type. Because of its prominence the dam at Keokuk may be taken. It has 119 openings, each 30 ft. wide, and the gates are 11 ft. high. Since the difference in high-water level above and below the dam is 20 ft. it is evident that the spillway openings easily could have been given that depth, which would have increased the discharge capacity $(20\sqrt{20}) \div (11\sqrt{11})$ or 2.45 times per linear foot. Instead, therefore, of a spillway length of $119 \times 30 = 3570$ ft., only 1460 ft. would have been needed. With rollers 100 ft. long—certainly within reasonable bounds in view of the maximum length already used—only fifteen rollers would have been required. The number of piers would then have been reduced from 118 to 14, and allowing a thickness of 10 ft. for each, 140 ft. would have been added to the length of the dam. Instead, therefore, of a spillway section 4278 ft. long (118 piers at 6 ft. each, plus 3570 lin. ft. of opening) the spillway with the roller type would have been only 1640 ft. long. It might be argued that a roller dam is more expensive than the one chosen. This is true if the cost per linear foot is considered, but the higher unit cost, even hasty figuring will indicate, would still show a very large saving in favor of the roller design. The operating advantage will also be heavily in favor of the rolling type.

It is apparent, therefore, that the rolling dam warrants far more attention that it has hitherto received from American engineers. That the new installations above cited will be an incentive for closer study of the type cannot be doubted, particularly when the intrinsic advantages are as great as here pointed out.

Economics of Consulting Engineering Advice

The development of central stations from small plants with limited local loads to large installations supplying energy to extensive industrial and residential areas has greatly increased the importance of engineering advice in connection with plant expansion. No general rule can be laid down as to the limits within which a local company organization profitably and wisely may handle its own technical problems without consulting outside experts, but where very large sums of money are to be invested in extensions of plant the presumption favors the cost of specialized advice. A specific case of this kind recently arose in a hearing before the Massachusetts Gas and Electric Light Commission, where the payment of $15,000 by the Fall River Electric Light Company to a prominent engineering house in New York in connection with a comprehensive enlargement of the company's generating plant was questioned by the board.

Vice-President Cabot called the attention of the commission to the fact that although the station is a comparatively new one, containing one 500-kw and two 1000-kw units, these machines are now too small to meet the conditions; that it would be folly to install further units of this range of size, and that the margins of profit on which the company now operates are so small that the cost of a mistake in engineering judgment might be serious, easily amounting to $100,000 in face of forthcoming expenditures planned to enable the company to be prepared to handle industrial power service in the large textile mills of the city; that it is necessary for the company to look forward to building a large station and that consequently the best possible engineering advice should be sought. The point was made that it would have been a mistake for the company's own staff to have attempted to decide the size and type of generators, boilers, stack and character of coal-handling equipment needed for a station with an ultimate capacity of 50,000 kw; that many important questions of pipe sizes and machine arrangement need to be settled far in advance of the completion of the work; and that the company has no desire to be faced five years hence with the necessity of discarding the equipment which it now proposes to install. Mr. Cabot further stated that the company secures the best results by combining the resources of its own staff with the advice of the outside engineering establishment; that the latter has a wider point of view, seeing developments all over the country, coming in contact with foreign practice, and at the same time being open to receive the fullest criticism and suggestion from the local organization's technical forces. He pointed out that the advice of these two sets of engineers contributes to the solution of problems without prejudice and is a form of insurance for the stockholders of the company. It was considered that the company, serving a city of about 100,000 people, was not large enough to maintain a staff of engineers who could render final judgment upon a development of the size contemplated, whose inauguration constituted a turning point in the company's engineering career.

Situations of this character are likely to become more common in American cities of moderate size in the future, and the limitations of a localized engineering organization with respect to technical prophecy can only be overcome by recourse to specialists skilled in forecasting lines of development, experienced in controlling initial investment in plant without sacrificing continuous and efficient development, and in comprehensive touch with the broadest resources of the engineering and manufacturing industries. In exceptional cases, no doubt, a central station will carry in its own organization an engineer of unusual vision and executive ability whose services will be practically self-sufficient for solving the problem of plant expansion most economically and satisfactorily, considering the future as well as the present. But in general, small industrial organizations, be they engaged in the production of commercial electrical energy, turbo-blowers or concrete mixers, cannot afford to maintain for only occasional use the degree of engineering skill and foresight demanded when financial commitments to the future are made on a large scale upon the basis of technical advice. Wearisome as it is to reiterate the need of co-operation in every branch of modern industry, it is important to emphasize that it is becoming more and more difficult for a single individual to render the most efficient judgment without combining his knowledge and sense of proportion with those of other men when large-scale engineering problems are under consideration. Modifications of individual opinion must be had to secure well-rounded development.

Putting the Small Filter Plant into Operation

Calibrating chemical feeders, making gages for coagulant tanks and rating wash-water pumps are all in the day's work of the resident engineer who remains through the "tuning-up" period of a filter plant. Ordinarily there is not a great deal of changing or remodeling to be done, but, such as it is, the final adjustment of the equipment means much for the future success of the plant and is often the means of developing novel operating kinks, hitherto untried, which give much greater satisfaction than the methods which practice has pretty well standardized.

Last June the Engineering Record printed an article on the difficulties encountered in putting the 20,000,000-gal. Minneapolis plant into operation. Last week a little 2,000,000-gal. plant at Albany, on the Pacific Coast, claimed attention. Some of the same difficulties were detailed because, whether large or small, there is required substantially the same equipment. A marked difference exists between the large and small plant when it comes to operation, for the small plant must usually be manned by lower-priced men and less skilled operators, who often have other duties to perform. Because of its smallness there is a tendency to develop "shelf-hardware" features, which, nevertheless, have their place, provided it is established that they will do the work. It is clear, however, that too many local conditions are variable and too many new kinks available to warrant standardizing the whole plant. It was fitting, then, for the management at Albany, when desired results were not obtained, to have the plant put in prime shape and have some one impart sufficient knowledge of the principles of filtration to the attendants to enable them to "swim alone."

Giving an elementary course in filtration in a classroom to university students at the receptive age is child's play compared with the Montessori methods necessary to get the same information into the head of the average filter attendant. Ordinarily, however, there is found an intelligent, resourceful mechanic who is more than willing to be instructed if the officials see fit to hire the instructor. Anyone who has ever taught such a man the use of a slide-rule can appreciate the eager interest which he takes in operating details.

That many of the older plants in the country could with profit be given a yearly "tuning up" is certain. Often these plants are in the hands of men who have entirely erroneous ideas of operation simply because they have their information from tradition. What can and cannot be done, according to tales told, and still get good water from some of these old, overworked, obsolete filters filled with sea gravel instead of sand frequently stretches one's credulity beyond the elastic limit. Bleach treatment has given some of them a new lease on life that is really marvelous. Engineers of State boards of health will detect some of the lapses, but those under whose direction the plant was originally installed may well spend a day now and then keeping in touch with the operation to see that the filters have, at least, friendly treatment. Last winter at a meeting of the Illinois Society of Engineers and Surveyors quite a stir was created by the statement that few engineers followed their work further than the design and construction stages. It was contended that it is a duty, neglected by too many engineers, to tell how new designs work, or fail. An appeal was made to the technical press to print articles telling how new features need modifying to make them useful. This is an indication that the time is ripe for engineers to pay more attention to supervision than has been done in the past. There is too much of the spirit of getting the job accepted and then letting the other fellow get on as best he can.

In the case of filters it is safe to say that in most instances the success or failure of a 2,000,000-gal. plant depends entirely on the intelligence and interest of a comparatively low-salaried attendant. It cannot be urged too strongly on officials that they teach these men, or have them taught, the principles on which the plant is constructed, and also the proper methods of handling the water supply of the city. Furthermore, "tuning-up" periods should not be too far apart, for operators change and corroded orifices need regraduating just as much as valves need repacking. The troubles of the valve are recognized; the diseases of the orifices are not.

Investigation of Present Economic Conditions

The National Civic Federation, from recent reports, has laid out a large job for itself in the examination of the industrial and social situation, with special reference to the question of private ownership and the method of distribution of wealth. Political economists have been studying such matters for many years, with somewhat discordant and unsatisfactory results. Whether the Civic Federation can add anything to the subject by its proposed researches remains to be seen. It must be admitted, however, that the plan of its investigation is somewhat fresh, in its inclusion of the industrial revolution of the past few decades. The older political economy was based broadly on the law of supply and demand, generally with the tacit assumption that supply and demand are essentially self-regulating, whereas the recent tendencies have all been in the direction of the artificial control of both insofar as elaborate and sometimes world-wide combination can influence them. The combination in restraint of trade is not altogether a new factor in political economy, but it certainly is one which until recent years has not been dominant, as it is now in many departments of industry.

The whole subject is a very large one, upon which there is room for extreme difference of opinion, and it would seem at first sight as though the work of the Civic Federation might better be directed to the solution of some of the pressing minor problems. For instance, in connection with modern methods of doing business—in food supplies, let us say, one of the most necessary things to ascertain is the effect of natural or induced scarcity upon prices. This has been to a certain extent worked out with reference to the price of wheat, in which case it has been found that the law appears to be roughly an exponential one, so that a given percentage of shortage produces much more than a proportionate increment of price. When one comes to look into the matter further there is very good reason to believe that the exponent differs for shortage and surplus, so that, for instance, while cutting the supply of a given commodity in two may raise the price fourfold, doubling the available supply will not decrease the price to one-quarter. In other words—if one may be pardoned an engineering simile—there seems to be a hysteresis loop of unknown magnitude affecting these results. It is on the existence of such a loop that the success of manipulation of food supply by storage probably depends. For example, eggs placed in cold storage in very large quantities in all probability command disproportionately high prices when dribbled out in time of scarcity; the higher prices are not compensated for by lowered rates in time of plenty. If they were so compensated the cold-storage game would be hardly worth the candle as a commercial proposition, and certainly nobody supposes that it is played for fun. The same sort of thing is brought about by one device or another in the case of many other commodities.

Another feature included in the work of the Civic Federation should be a study of the distribution of costs and values along the complicated line of distribution followed by most articles of trade, and the effect of these complications on the public. To cite a pertinent example which will appeal to every householder irrespective of business, why it is that apples may sell in a city market at $3 by the barrel when they actually do not bring enough at the source, within 100 or 150 miles of the city, to pay for picking and barreling, so that thousands of barrels rot on the ground? A little time, too, might be spent in the investigation of the economic relations of throwing into the sea a ship load of sound tropical fruit lest its advent should break prices.

These minor things are far easier to investigate than the great fundamental matters which lie behind the sources of wealth, and the remedies found could be far more easily applied. The Civic Federation will be doing an admirable thing if it can successfully deal with even some of the simpler problems.

Frost Damage to Montreal Filters

Concrete Piers, Floors and Groined Arches, Unprotected during Winter, Crack and Spall — Repair Methods and Report by Board of Experts

By FREDERICK E. FIELD
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At the end of the summer season for outside construction contractors and engineers should give serious consideration to the proper protection of uncompleted work against frost action during the winter. Neglect in so doing is too frequently followed by serious damage, necessitating delay and costly repairs. During the winter of 1912 the filtration works under construction for the city of Montreal suffered considerable damage. Frost action on the partially completed structures produced many unusual results, which form the main theme of this article.

Contracts

A general description of the design and main features of the plant was published in the Engineering Record of Sept. 2, 1911, page 280. The work is divided into several contracts. Under contract 1 will be furnished the pumping machinery, blower, cranes and other equipment necessary for a complete installation, all of which will be located in a building adjoining the filtered-water reservoir. These pumping units are required to lift the water from the supply conduit to the prefilters, from which it will flow by gravity through the final filters and filtered-water reservoir to the municipal pumping station.

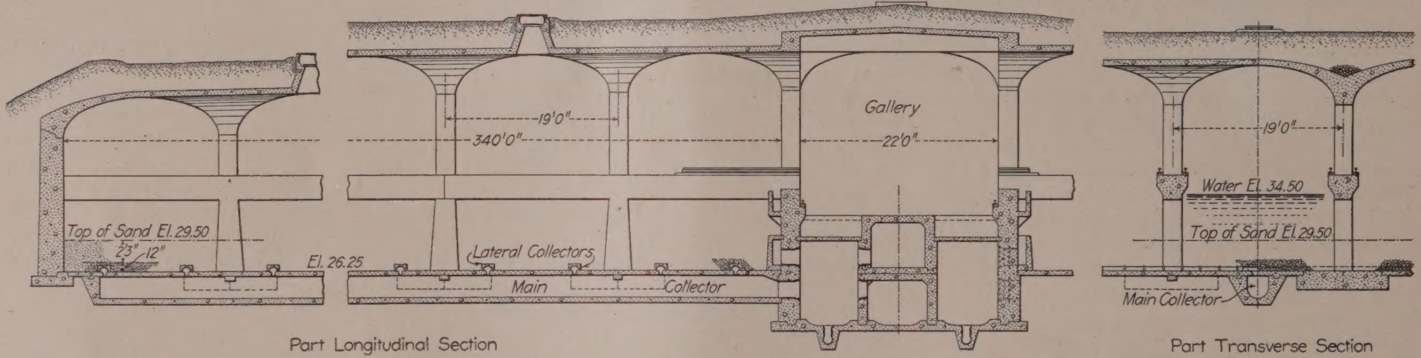


Fig. 1—Longitudinal and Transverse Sections through Filters, Showing Beam Runways for Sand-Washing Machine

Contract 2 comprises the final filters and appurtenances, which will later be described in detail, as it was upon this contract that the damage by frost occurred.

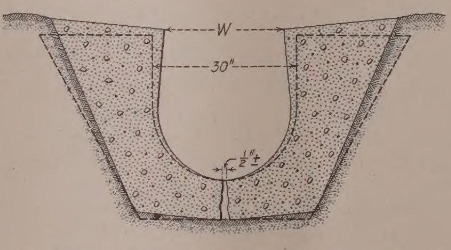


Fig. 2—Type of Collector Failure

W was originally 30 inches.

Contract 3 includes sixteen prefilters, each with a normal capacity of 3,250,000 imp. gal., a filtered-water reservoir with a capacity of 6,000,000 imp. gal., and various supply and discharge conduits.

Contract 4 will include the filtration pumping station, wash-water tower and the superstructures of the prefilter operating gallery and gate houses.

The general arrangement of the entire filtration works is shown in the issue of this journal previously referred to.

Final Filters and Appurtenances

There are sixteen final filters, each having a net filtering surface of ⅜ acre, arranged in batteries of eight on either side of an operating gallery. The normal capacity of the filters is 50,000,000 imp. gal. per day when operated at a rate of 10,000,000 gal. per acre per day. The equipment is complete, including lighting, heating, and ventilating systems and sand-cleaning machinery.

Some of the more important items included in the work are: Earthwork (excavation and embankment), 100,000, cu. yd.; concrete, 34,000 cu. yd.; steel reinforcement, 1,000,000 lb.; cast-iron pipe and specials, 112 tons; wash-water gutters, 16,300 lin. ft.; filter material (sand and gravel), 35,000 cu. yd.; washing machine track system, 33,700 lin. ft.

Features of Design

The filters are each 55 x 340 ft. in plan, to accommodate Blaisdell sand-washing machines, which will run from the gallery to the extreme end of the filters upon tracks carried by concrete beams between the filter piers. The filters, Fig. 1, are covered by groined arches, supported on piers spaced on 19-ft. centers. The floor is flat and so constructed that the pier and wall loads are carried on reinforced pier and wall bases, entirely independent of the regular unreinforced 5-in. floor.

The main collectors run longitudinally the entire length of the filter and below the floor slabs.

In general the walls and wall bases are built in sections 57 ft. long, and the walls are built to their full height at one pouring. The track beams are built in alternate sections, each 38 ft. in length, with expansion joints on the center line of every second pier.

The gallery is 454 ft. long and 22 ft. wide and contains the prefiltered and filtered water conduits and the necessary piping connections to the filters. The side walls carry tracks upon which a transfer table for conveying the washing machines from one filter to another will be operated.

Foundations

The entire foundation is a secondary deposit of material previously eroded by glacial action, and deposited by running water. The surface layer is yellow, while the underlying material is a grayish blue. An analysis of the soil shows that it consists of fragments of rock

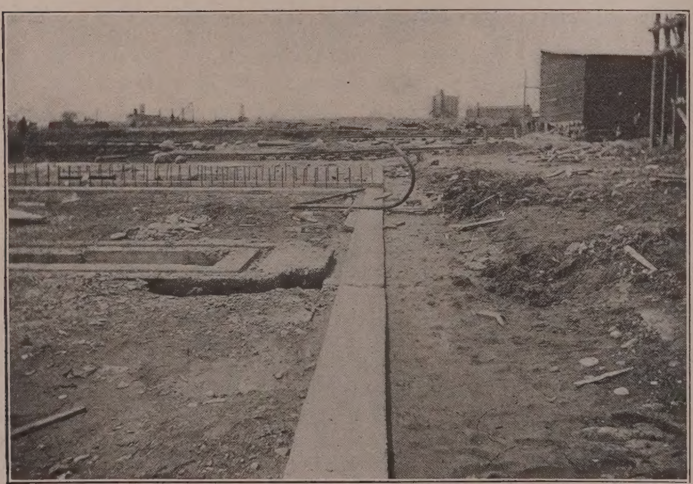


Fig. 3—Damage to Concrete at End of Collector

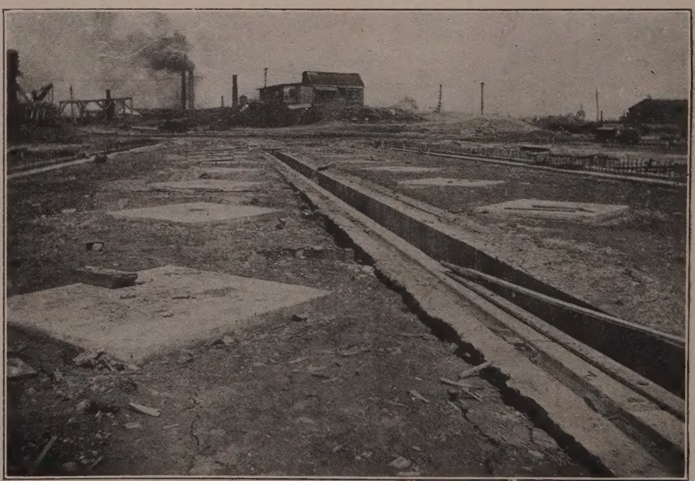


Fig. 4—Pier Bases and Crevasse along Drain

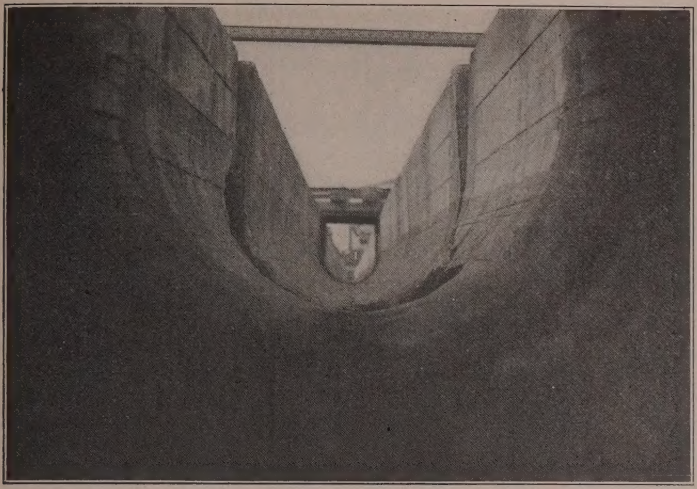


Fig. 5—Cracks in the Main Concrete Collector

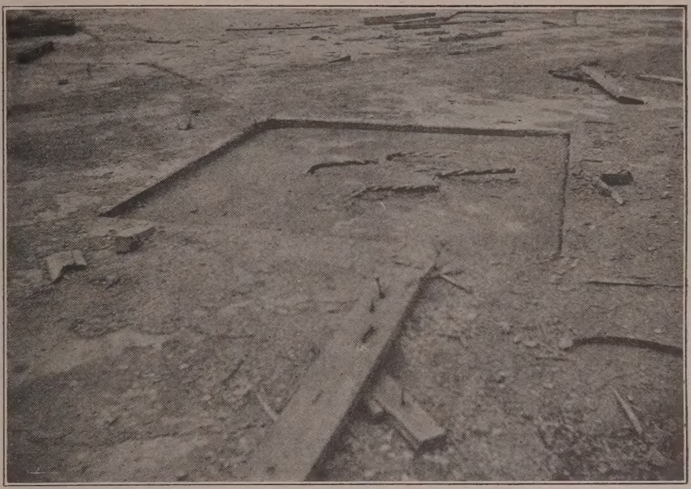


Fig. 6—Condition of Concrete Floor Slabs

and rock powder graded from coarse to very fine particles, all of which under the microscope are found to be sharp and angular.

Actual tests of the bearing power of the soil have shown that it will sustain loads of 3 to 5 tons per square foot, with only a slight compression of the soil even when saturated with water. In some of the tests the apparatus was surrounded by water to a depth of 8 ft. for two weeks without any appreciable settlement under the ram. As the maximum load to be carried by the soil does not exceed 2½ tons per square foot, these tests proved the soil to be entirely adequate for the loads to be imposed.

Table 1—Meteorological Data for Montreal

Temperature	Dec., 1912	Jan., 1913	Feb., 1913	Mar., 1913	Average	Dec. (d)	Jan. (e)	Feb. (e)	Mar. (e)	Average
(a) Mean	24.38	22.68	11.81	28.76	21.91	19.03	12.68	14.88	25.18	17.94
(b) Maximum	31.05	31.72	20.70	36.18	29.90	25.81	20.92	22.85	32.13	25.40
(c) Minimum	16.87	14.62	3.96	21.07	14.10	12.02	4.73	6.93	17.81	10.40
Totals in inches					Totals					Totals
Rainfall	1.83	2.56	0.02	4.54	8.95	1.30	0.87	0.78	1.50	4.45
Snowfall	14.10	22.50	22.40	19.00	78.00	24.90	29.74	25.37	21.40	101.41
Precipitation	2.91	5.08	2.79	6.42	17.20	3.75	3.73	3.29	3.79	14.56

(a) Mean of bi-hourly readings taken from self-recording instruments; (b) and (c) averages of the maximum and minimum temperatures for each day of the month; (d) thirty-eight years' "mean" for and including this month of 1912; (e) thirty-nine years' "mean" for and including this month of 1913.

Work Completed Before Winter of 1912

The area covered by the sixteen final filters had been stripped of top soil, the entire area leveled off and brought to the subgrade of the filter floors, and embankment placed to a depth of from 4 to 6 ft. along the completed outside walls. In making the necessary fills to bring the ground up to the subgrade of the filter floors the best clayey material from the excavation of the filtered-water reservoir was used, all such material being placed in 4-in. layers and thoroughly rolled. There were only two comparativey small areas where there was any considerable depth of filling below the pier bases. One of these areas required a maximum fill of about 3 ft. and the other a fill of 4 ft. at the lowest point of the bowl-shaped depressions. Only 8 per cent of the pier bases is on fills exceeding 6 in. in depth and more than 80 per cent of the foundations is upon original ground.

The main collectors and pier and wall bases were completed in all of the sixteen filters. Floor slabs were completed in eleven filters, no slabs having been placed in five filters on the north side of the gallery. All outside walls, practically all dividing walls, and all but a few track beams were completed on the south side of the gallery; but very little work of this character had been done on the north side.

The vaulting had been completed over a little more than two filters.

Protective Measures Neglected

All work was discontinued about the middle of November. The contractor did not make any provision for protecting the foundations from frost action, although he had been officially notified that serious damage might result if the work was not protected. The entire work could have been flooded with water to prevent the freezing of the ground below the foundations. This would have been an easy and cheap method of protection.

As an indication of the probable efficiency of such flooding it is instructive to note that the completed sections of concrete floor in the filtered water reservoir were in a depression which allowed surface drainage to collect to a depth of from 3 to 5 ft. which protected the work so well that there were no freezing of the ground below the floor and no resultant damage or lifting by frost.

Range of Temperatures

The range of temperatures, together with the amount of rainfall and snowfall, is shown in the table of meteorological data. These statistics are based on reports from the McGill College observatory at Montreal. In comparison with the corresponding months of previous years the weather conditions were unusual in the following particulars: (1) The average temperature was higher by about 4 deg. (2) The total rainfall was in excess by about 100 per cent. (3) The total snowfall was less by about 22 per cent. There were no long continuous periods of low temperatures, and changes from low to moderate temperatures were frequent and sudden, which would tend to increase frost action.

Early in March, 1913, after the snow and ice had largely disappeared from the site, it was found that the disturbance by frost was

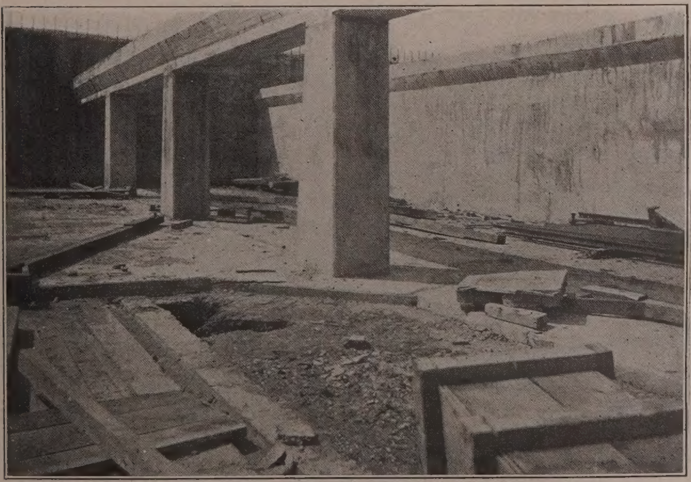


Fig. 7—Example of Cracking of Pier Base

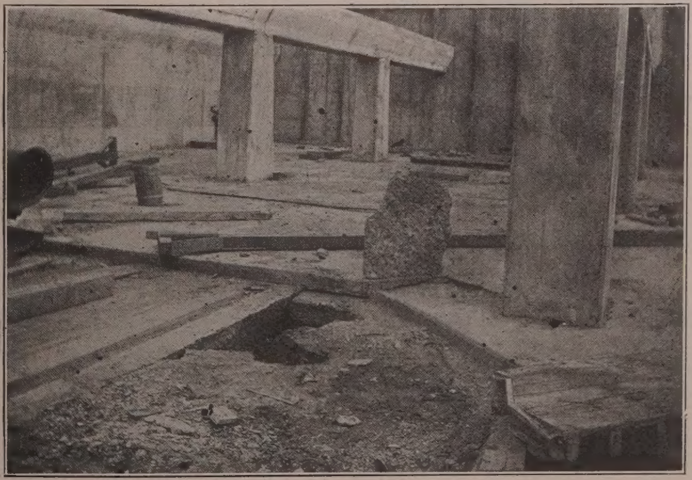


Fig 8—Spalled Concrete Removed from Floor

general and much damage had occurred. The main collectors and floor slabs were raised and many of them badly cracked. The pier bases were raised and tipped and many were also "spalled" where adjacent floor slabs abutted. The wall bases were generally raised and some of them broken completely through by their unequal vertical movements. The track beams were raised and some of them were cracked from the varying movements of their supports. The dividing walls were raised and tipped and the expansion joints between sections were opened considerably more at the top than at the bottom of the walls. The roof groins were unequally raised along the outside barrel arches and some of them were displaced with respect to the adjacent sections of vaulting.

As spring advanced and the frost left the ground further changes took place. The floor slabs settled and the widths of the cracks were reduced. The pier bases practically returned to their original position and the "spalling" along edges in contact with floor slabs became more general. The wall bases and walls also returned toward their original elevation, with a resultant closing of the cracks at expansion joints. The track beams followed the downward movement of the pier bases; some of the cracks closed and others developed as the result of unequal movement. The roof groins returned to place with some minor cracking at the springing line, and in some cases the upper piers were slightly tipped.

While the damage sustained by frost action was serious, the fact that this damage was no more extensive is not only conclusive proof of the relative flexibility of the various structures but also that the work, as designed, was able to suffer great punishment without complete failure. It is interesting to note that the lifting of the foundations by frost was less on those areas where filling had been done than where the footings were on original soil. This may be explained by the fact that the filling was of selected material, placed in thin layers and thoroughly rolled, with the result that it was more compact and contained less voids for the entrance of moisture than the original ground.

A detailed description of some of the damage follows:

Main Collectors.—The general type of failure is indicated in Fig. 2. The pressure of the freezing and expanding ground forced the sides inward and produced a longitudinal crack throughout the length of the invert, and also raised the collector from its original elevation. The width of the collector was originally 30 in., but the sides were forced together until in some instances the width at the top was reduced to 23½ in. Figs. 3 and 4 show the lifting of the collectors, while Fig. 5 illustrates the forcing together of the sides.

All cracked main collectors have been replaced, while collectors lifted and not cracked in the invert are being repaired by forcing grout into the space below the collectors with sufficient pressure to fill all voids.

Floor Slabs.—The floor slabs, while lifted and frequently badly cracked by the frost, have not returned to their original positions to the same degree as the pier and wall bases (see Fig. 6). Inspection shows that in many instances the ground has settled to its original elevation, leaving the slabs partially unsupported and with a considerable space between the bottom of the slabs and the earth. Badly cracked slabs will be replaced and others may be repaired by forcing in grout under pressure to fill all voids and cracks.

Pier Bases.—The upward and return movements of the pier bases varied considerably in various parts of the work. Levels were taken on each of the four corners of the pier bases at frequent intervals to determine their movement. The figures in Table 2 show the recorded movement of the pier bases in Filter 16, which may be considered as typical.

TABLE 2—MOVEMENT OF PIER BASES, FILTER 16

Date	Average elevation, feet East Row	West Row
Summer, 1912	26.26	26.25
April 5, 1913	26.41	26.37
April 30, 1913	26.31	26.29
June 5, 1913	26.28	26.27

It will be noted that the greater return movement took place between April 5 and April 30, during the time the frost most rapidly left the ground, and that on June 5 the pier bases had nearly resumed their original elevations.

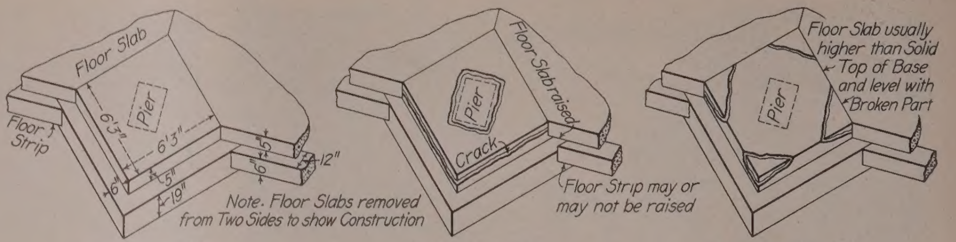

Floor Slab
Pier
Floor Strip
6'3"
6'3"
5"
6"
5"
6"
12"
19"
Note. Floor Slabs removed from Two Sides to show Construction
Pier
Floor Slab raised
Crack
Floor Strip may or may not be raised
Floor Slab usually higher than Solid Top of Base and level with Broken Part
Pier
Original Condition
Type A Failure
Type B Failure

Fig. 9—Type of Failure of Pier Bases—Floor Slabs Removed from Two Sides

SPALLING OF PIER BASES

Possibly the most peculiar result of frost action was the spalling of the pier bases. This spalling, or horizontal cracking of the pier base above the shoulder upon which the floor slabs rest, was typical of a large number of pier bases against which floor slabs had been placed. In some instances the floor slabs were not so tightly wedged against the pier bases and no spalling of the pier bases was caused by the different relative movements of floor slabs and pier bases. Figs. 7, 8 and 9 illustrate well the nature of this spalling. Fig. 8 shows a portion of the spalled concrete removed, and it will be noted that the stones in the concrete are sheared by the great lateral pressure that obtained.

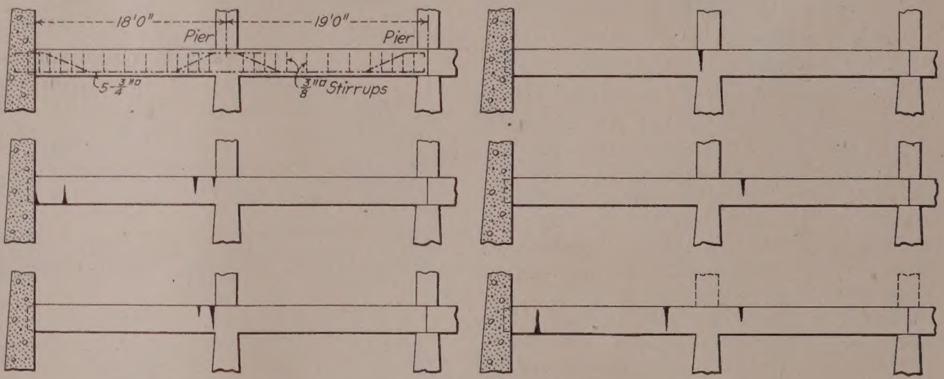


Fig. 10—Location of Cracks in Beams which Support Runway Tracks

It was found by investigation that the greater number of pier bases were thus spalled between April 5 and 30, which, as shown in Table 2, was the period during which the frost left the ground most rapidly and the greater movement of the pier bases occurred. There was also a similar spalling of some of the wall bases against which the floor slabs had been placed.

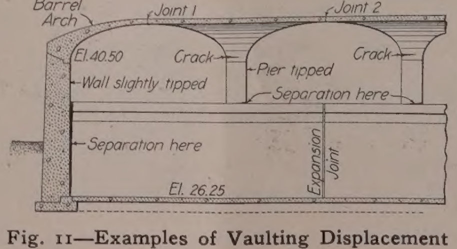


Fig. 11—Examples of Vaulting Displacement

Track Beams.—The track beams opened at the expansion joints, as was to be expected, but the openings were not of equal width, the maximum openings being usually located at the expansion joints nearest the north and south walls of the filter. This was probably due to the fact that the outside walls were not appreciably, if at all, displaced by the frost. Their footings were somewhat protected and the raising of the adjacent piers would make the opening of the expansion joints more pronounced where one end of the beam was rigidly held than in the case of intermediate beams, where all the pier supports were lifted more or less uniformly. The cracks in the beams also occurred in those bays adjacent to the end walls, as indicated in Fig. 10.

In March cracks were seen on the top of the beams near the pier and in the bottom of the beam near the outside wall. Later as the frost went out of the ground and the pier bases settled these cracks closed so as to be almost invisible and other cracks developed in the tops of the beams near the rigid outside wall.

Roof Groins.—As in the case of the track beams, the greatest visible movement of the roof groins was adjacent to the outside walls. The outside wall and barrel arch remained stationary, while the lifting of the adjacent pier bases by the frost raised the upper piers and roof groins above the plane of the barrel arch, and in some cases above the adjacent roof groins. This displacement varied from a very small amount to 1½ in. When the frost left the ground the pier bases settled to their original position and the roof groins also went back to place, with some slight cracking of the arches near the springing line and a slight tipping of the upper piers, all of which became less apparent as the season advanced.

In several cases when the roof groin and adjacent barrel arch were built monolithic cracks developed approximately parallel to and not far distant from the crown of the arch, and these cracks also closed as warm weather approached. The movement of the roof groins is indicated in Fig. 11.

Controversy Over Damaged Work

In the spring of 1913, after it was apparent that the work had been seriously damaged, the contractor advised the city that he had investigated the work with a number of well-known engineers and that they had found "that the disturbance by frost is a minor matter compared with the subsidence of practically the entire outside walls and various parts of the structure where footing loads are appreciable. The subsidence of the walls would indicate that the structural design is inadequate for the bearing power of the soil, which, while having the general appearance of clay, is in reality a finely divided sand, stable enough when dry but unstable when wet, it being in fact a quicksand."

The work was also carefully examined by city and filtration engineers, who found that the soil was adequate for the loads to be imposed, that the design was in accordance with the best modern practice, and that the damage was wholly due to frost action, for which the contractor was responsible, as it was his specified duty to properly protect his work.

The city also appointed, Messrs. J. A. Jamieson, P. W. St. George and F. A. Barbour, who had not in any way been previously connected with the work, and instructed them to report on the adequacy of the soil and design, to determine the cause of the damage, and whether there had been any subsidence of the structures as claimed by the contractor.

Board of Experts Reports

The findings of this expert board of engineers are as follows: (1) That the action of frost on the soil under foundations has been the cause, directly or indirectly, of all damage which has occurred.

(2) That settlement has not been a factor in causing damage up to the present time.

(3) That the design has good precedent, is adequate, and, if properly carried out, the resulting structure will be well adapted to the intended purpose.

(4) That the soil has and will have under operating conditions sufficient bearing capacity to support the loads to be imposed, and that such minor settlements as will occur, either on soil in natural position or in filled areas, will be, as shown by the tests made, well within the limit to which the structure may be subjected, without detrimental effect.

(5) That the design and method of construction of the arches are in accordance with good practice, and the superstructure is of a type which does not require absolutely equalized foundation loads, this being indicated by the abuse which the superstructure has already withstood without serious final damage.

(6) That the walls, piers and structure which were raised have now practically returned to original position, and, except for some spalling of the masonry and the cracks in the division walls, will be, when the full load is on the roof, in practically as good condition as before the movement.

(7) That in addition to the replacement of the damaged collectors, which has already been undertaken by the contractor, the important work necessary to return the structures to a condition practically equivalent to that which would have obtained if no movement by frost had occurred, includes the replacement or repairs of lifted floors, slabs, and collectors, the repairs of cracks in the division walls, gallery floor and conduit, and the replacement of the retaining wall back of the gallery walls.

(8) That the repairing by forcing cement grout under pressure below structures which have been lifted from a good bearing on the soil should, preferably, not be undertaken until after the full load to be imposed by the filling over the roof is on the supporting members, but that such grouting of space between masonry and the underlying soil should be completed before water is admitted to the filter units for any purpose.

(9) That since the raising of floors and collectors by frost has introduced abnormal conditions, it is essential that all vacant space be filled completely, or the lifted structures be replaced, and that the success of such filling of voids by grouting as may be attempted can only be proved by a test for leakage of the completed filter unit.

(10) That the fulfilment of such test of leakage may be accepted as proof that no damage affecting the working capacity of the filters had resulted from the movements occasioned by frost during the past winter.

After the submission of this report the contractor acknowledged the damage to the work had been entirely caused by frost, agreed to repair all damage at his expense and had again resumed the work of construction.

The filtration works form but a part of an extensive program for the enlargement and improvement of the water supply for the city of Montreal, all of which is under the direction of Mr. George Janin, chief engineer of the city, and Mr. T. W. Lesage, engineer superintendent of waterworks. Hering & Fuller of New York were the filtration engineers in charge of design. Mr. Norman M. McLeod is the contractor for contracts 2 and 3.

Collapse of Orpheum Theater

Bureau of Buildings Files Report of Investigation into Failure of Steel Structure

The collapse on Jan. 22, 1913, of a large portion of the steel roof and floor of the 150 x 78-ft. Orpheum theater building at Eighty-seventh Street and Third Avenue, New York City, was investigated by the Bureau of Buildings. The report submitted to Mr. Rudolph P. Miller, superintendent of buildings, by Mr. Alfred Ludwig, chief inspector, and filed Nov. 10, gives a description of the structure, the conditions under which it failed and the result of the collapse. An abstract of the report follows.

General Description

The greatest length of the steel roof trusses was 85 ft.; some of them were supported on H-shape columns made of four 6 x 4-in. flange angles and a 12-in. web plate with a practically unsupported length of 64½ ft. These columns had a cross-sectional area of 21.22 sq. in., a radius of gyration of 2.54 in. and a value for L/R of 283. At the time of the collapse they carried an estimated load of about 166,000 lb. each, equal to 7823 lb. per square inch. According to the column formula required by the Building Code, the safe load of a steel column is $P = 15{,}200 - 581\ L/R$, the ratio L/R being limited to 120. By this formula the safe working load for this column has a negative value. This is significant not for the actual result—which is beyond the proper use of the formula and merely shows the limited efficiency of a column with these proportions—but because it indicates the undesirability of loading slender columns heavily. This column and others have foundations on rock.

The balcony framing was of ordinary cantilever type, with steel floorbeams and heavy concrete floor slabs. The steel roof trusses supported the I-beams and reinforced cinder concrete slabs 4 in. thick. The structural steelwork had been completed, and all of it was so badly injured as to require removal.

Failure of Steelwork

A careful examination disclosed no defective materials or workmanship that might have caused the collapse. The foundation showed no displacement which might have caused the failure, although it had been thought that such result might have been produced by blasting in the vicinity.

The collapse was evidently caused by the failure of the steelwork, but whether this was due to faulty design or overloading has not been positively determined. There were three points at which initial failure might have occurred—truss supporting balcony cantilevers; balcony girder columns, and roof truss columns.

The bottom chord of the truss carrying the balcony cantilevers was stressed about 21,800 lb. per square inch and broke without distortion, producing a sharp fracture through the rivet holes, and showed neither elongation nor reduction of area. On account of the moderate load, the appearance of the fracture and the lack of buckling in the top chord this method of failure is considered very improbable. The fracture had the appearance of being produced by a sharp, quick blow. Test pieces cut from the bottom chord showed a minimum yield point of 40,000 lb. and ultimate strength of 48,000 lb. in tension.

After the collapse, the balcony truss columns were found bent and distorted, and it was suggested that the serious overloading of the balcony cantilever girders by the heavy concrete floor they supported produced a bending moment in the columns causing their failure. This view was believed to be erroneous because the maximum girder stress at the time of failure was only about 24,000 lb. and because the deflections of the balcony girders and cantilevers were too small to produce a very heavy eccentric moment in the columns.

The long columns supporting the roof trusses were theoretically inadequate for their loads, as previously noted; but an inspection of the collapse failed to show conclusively any such crippling or buckling as would be expected, thus leaving serious doubt as to whether they failed under the load. They were bent only at the points of attachment to their inadequate bracing, and while various indications point to their failure as the initial cause of the collapse, absence of any pronounced crippling or buckling is a serious objection to this theory.

Conclusions

The columns supporting the balcony roof and trusses were insecurely braced, and more attention should be paid to bracing H-shape columns.

The H-shape columns are considered unsatisfactory on account of the small radius of gyration in comparison to the sectional area. H-shape columns are considered unsatisfactory because only one leg of the angle is fastened to the web and the other leg is wholly unsupported both in rolled and in built-up sections.

In columns of the box type each part or shape mutually supports and stiffens the other parts, and the least radius of gyration is much greater than it is for an H-shape column of the same cross-sectional area.

Port Organization and Development—Part I

Discussion of the Various Systems in Vogue and Argument for Commissions Representative of the Numerous Interests Involved

A YEAR ago authorities representing the various ports of the United States organized to study "the new science of port management." The importance of the subject is coming to be realized more and more. The newly elected president of the Institution of Civil Engineers, Mr. Anthony G. Lyster, adopted the above-mentioned topic for his inaugural address, delivered Nov. 4. While his views are based chiefly on conditions in the United Kingdom, he refers to the ports of other countries and draws conclusions that will be seen to be generally applicable. These notes are taken from his paper:

In islands such as Great Britain and Ireland anything that affects the efficiency and development of their trade outlets and inlets touches vitally the welfare of the whole kingdom. The densely populated condition of the country, the fact that the land cannot provide sufficient foodstuffs for the people, the great mineral wealth which the country possesses, the highly trained and organized condition of its industrial population, which requires large supplies of raw materials from outside sources for its operations, combined with the system of free imports which prevails to so large an extent, all necessitate a very efficient system of port administration in order to give free scope to and insure the best development of the commercial and industrial activities of the nation. The authorities, therefore, to whom the maintenance and development of the ports are committed should be bodies not only capable of bringing special commercial knowledge and sound judgment to bear on the variety of problems with which they have to deal, but also able to take a broad view of their responsibilities, and to recognize that national and imperial as well as local interests are involved in the successful administration of their charge.

Classes of Ownership

In general, docks and harbors are owned (1) privately, (2) by public dock companies, (3) by railway companies, (4) by municipal corporations, (5) by commissions, and (6) by governments. The constitution of the various authorities controlling these different undertakings seems to be based mainly on two broad principles absolutely opposed to each other. One is that those who have found the money for the provision of the accommodation should be allowed directly to control its development, without representation of the traders using it; the other is that those who have provided the funds for the creation of the port should have nothing whatever to say to its management or control, which is vested almost entirely in the representatives of the traders, who at the first blush might be considered more immediately concerned in the promotion of their own prosperity than in that of the trust which has been vested in them. This fact clearly demonstrates that the constitution of these variously governed ports, in Great Britain at all events, has not been based on any common conception or standard of suitability. The effect has been to create a large number of ports in which the constitution of the authorities presents a great variety of qualifications.

Dock and harbor systems under private or company ownership have come into being, as a rule, to develop natural resources such as coal or other minerals. Their traffic is more concerned with a local export trade of that character than with a general trade of imports and exports affecting extended and widely scattered areas of the country, and numerous sections of the community with divergent interests. The effect, therefore, of the management and development of such properties cannot be regarded as having the same extended importance as that of undertakings which deal with many and varying classes of trade having wide-reaching ramifications and affecting the welfare of many and important industrial communities.

Perhaps the one important exception to this description of docks owned by companies is the Manchester Ship Canal Company's undertaking, whose trade is of a general character. The particular circumstances of this undertaking, as well as the huge capital which was necessary at the very outset for its construction, make it so exceptional a case as to prevent any fair deductions being drawn from it on the subject of the successful working of a large general dock system by a private company.

Railway Ownership

Docks owned by railway companies have also largely come into existence for the purpose of developing some particular trade, such as coal or other minerals, and in these circumstances the question of the suitability of their organization to public or national requirements does not seem to involve any special consideration or raise any points that have not already been dealt with in the case of docks owned by public dock companies. There are, however, important exceptions to this description of trade carried on by railway-owned docks, and at Southampton and other ports there exist groups of docks owned and managed by railway companies which are engaged in more general forms of trade. In such cases these docks come into competition with other systems owned and worked by specially constituted authorities, whose functions are confined to the docks alone, and objections have been frequently raised to railway ownership that it gives rise to unfair competition which it is difficult for those bodies who do not own railways to meet effectively.

Objections, too, have been raised by other railway companies to rival companies owing dock property, and so being able, by lowering the dock rates, to divert traffic to their lines. The primary interest of a railway company is to attract as much traffic as possible to its line, and to this end it may regard the docks largely as feeders to the railway system, and may encourage their use by trades which are good customers in that way, at the expense of those which do not require to use the line. It may also, by charging specially low rates for dock accommodation and recouping the deficit from the revenues of the railway, compete unfairly with dock authorities whose business is limited to the provision and management of that class of accommodation.

On the other hand, however, the railway companies, with their wide knowledge of trade, their power of combining dock and railway service, their ability to link up various trade centers and collect and distribute goods throughout the country, are frequently the only bodies who can successfully initiate and develop a port and a trade. This argument, too, may extend in a measure to general as well as special trades, for although the port may, in the first instance, be started in the interest of some special trade, such as a group of coal mines in the hinterland, the tendency will be as it develops to create other important general interests and a local population with subsidiary trades necessary for their support. Moreover, railway enterprise is always on the lookout for fresh opportunities of creating and fostering trade, and so is frequently the only competitor in this particular field of commerce; and it would be a misfortune to limit unduly the enterprise of such important agents in this class of commercial development.

Municipal Ownership

As an example of a port owned by a municipal body, that of Bristol is probably the most prominent and important. About the thirteenth century the Corporation of Bristol undertook improvement works, and since that time, and in the early days of the port, many schemes were initiated and put forward by different bodies. The initiative was frequently, if not generally, the result of private enterprise. In these old days the position and importance of the port of Bristol was second only to that of London, whereas it has in more modern times been surpassed by a great number of ports, and now stands, in respect of the volume of its tonnage, only about sixteenth on the list.

It would be unfair to attribute this want of expansion in later years solely to the fact of its being controlled by the municipality, but this fact is not to be lost sight of. Other circumstances, such as the growth of manufacturing industries in various parts of the country, and the large populations which they have attracted, have undoubtedly mainly contributed to the growth and prosperity of all ports. In this respect Bristol's position compares very unfavorably with ports like Liverpool, Hull and Glasgow, which form the outlet and inlet for great commercial industrial centers and iron, coal and shipbuilding districts. There are no such commercial or industrial areas as these behind Bristol, and it was not to be expected that its growth as a port could be anything like commensurate with that of others infinitely more advantageously situated. On the other hand, it may fairly be argued that the fact of its being controlled by a municipality is not in its favor. Under this arrangement it has the credit and backing of the municipality to lean upon; but as this support involves a tax upon the municipal ratepayers it is doubtful whether a policy of receiving with one hand and giving away with the other compensates for the drawbacks of a system of control which is unsatisfactory in many other respects.

Commission Management

For the most part commissions consist of ship-owners and merchants concerned in the trade of the port, but representatives of other bodies are introduced in practically all cases in greater or less proportion. The most important of these outside representatives are generally those from the local municipal bodies, and in some cases their representation is very considerable. The advantage of the municipal representative element in such a body is that the port gets the benefit of the financial support and credit of the municipality, these bodies being able, as a rule, to borrow money at a lower rate than a dock authority. Where, too, the interests of the port and town harmonize the combination of representation is an advantage and promotes smooth working. On the other hand, the element of politics is a strong factor in the selection of municipal candidates, and their policy and influence therefore can never be freed from that curse attendant upon the system of popular government—namely, party bias.

There are other important disadvantages in the association of municipal representation with harbor management. One of these is the tendency of corporations to regard all large industries and undertakings within their jurisdiction mainly as contributors to the municipal revenues, and to use them for that particular purpose. Such a policy is, in most cases, entirely opposed to the larger interests of the port and will be a hindrance to its development. It may also lead to the expenditure of money on the port in a manner conducive to the welfare or supposed interests of the town community rather than to that of the port generally. Again, in most large general ports a very large proportion of the trade is connected with industries in the interior, and it is essential for their prosperity that the transit of materials to and from such areas should be conducted as directly and cheaply as possible, with the smallest possible amount of handling and local charges; and the promotion of this class of traffic is not in the direct interests of the municipality.

Municipal authorities, too, have strictly defined limits of jurisdiction, and the question of providing accommodation outside those limits may arise at any time. The case has arisen in more than one instance of the most suitable position for extension from the point of view of the port lying outside the municipal boundaries, and municipal influence being exercised in favor of sites within its limits.

It is significant that some of Great Britain's largest ports, such as Liverpool and Glasgow were all in the early stages of their history under municipal control, and that it was deemed expedient to change their constitutions and convert them into trusts, or in the case of London to sell the city's interest to dock companies.

Continental Management

Continental practice seems to have developed upon quite different lines from those prevailing in Great Britain and to have relied chiefly upon Government or State initiative and control, rather than upon individual and local enterprise. The general method of managing ports still in vogue in Continental countries is for the State to investigate port projects, usually prepared by an engineer appointed by itself, and, if approved, provide the funds for their execution. In most of such cases, its benefactions in this respect are supplemented by subsidies provided by the municipal corporation or chamber of commerce at the port concerned, for which they receive certain port charges. There are several exceptions to this general rule—notably Antwerp, Amsterdam, and Rotterdam, where the docks are owned by the cities; Bremen, where the harbor belongs to the city and the docks are under the control of the State; Copenhagen, Barcelona, Bilboa and Genoa, which are all administered by commissions.

In the case of ports in the early stages of development such a system of state initiation and control may have distinct advantages. It is difficult to imagine, however, that in the public interest, as in the case of ports which are naturally trade inlets and outlets, and where consequently trade flourishes on its own merits, such a system can be satisfactory, or can produce such good results as the system of self-support which prevails in England.

(To be concluded)

Concrete Flume That Did Not Fail

Landslides Fill Congdon Ditch, Washington, Causing Water to Overflow and Scour Out Foundations, Leaving 41 Feet of Flume Unsupported

In contrast to all accounts of all too frequent failures of modern structures, whether of steel or stone, the following has to do with a reinforced concrete flume that is still doing duty, although on at least two occasions partial or complete failure of certain portions of it might well have occurred, it is thought, without reflecting any discredit upon its design or construction. This structure is the reinforced concrete flume commonly known as the Congdon Ditch, which has its intake in the Naches River about 9 miles above the city of North Yakima, Wash.

The flume, which is about 6 miles long, and conveys water for irrigating land controlled by the Yakima Valley Canal Company, was originally built in 1892 as a timber flume and earth ditch. The route followed, however, is over such rough ground that 5½ of the 6 miles were wooden flume, only about half a mile being stable enough to support an earth ditch.

Steep Sidehill Location

For practically its full length the line runs along a steep hillside just under a bluff composed of loose boulders of all sizes. The geologic formation is an Andesitic bluff overlying cement gravel. The debris from the bluff, fallen over and lodged on the somewhat easier slope of the cement gravel, is mixed with a large amount of volcanic ash. At some points there are large deposits of this ash, which has the fineness of flour and the peculiar property, it is said, of decreasing in volume when wet. This ash has been for ages subject to the action of the winds, apparently, which have picked it up as dust on the west side of the ridge and carried it over to be deposited among the boulders below the volcanic bluff on the east slope. The flume winds along the east slope and has to depend upon this unstable material for its foundation.

For about three-quarters of its total length, it was found practicable to build the flume on an earth bench about 7 ft. in width. When the slope exceeded 45 deg., or where there was danger of slides, a culvert type was constructed and then covered with earth. At points where a foundation in earth was not feasible, such as in passing around bluffs or in crossing gulches, the flume was supported on 8-in. reinforced concrete piers extending to solid rock. No expansion joints were used, but provision for contraction was made by the use of longitudinal reinforcing steel, which, it was thought, would also give the structure added strength.

Type of Flume

When built on a bench excavated in earth the bottom of the flume was made 4 in. thick. The inner wall was also made 4 in. thick and leaned toward the hill on a slope of ¼ to 1, while the outer wall was 6 in. thick at the bottom and 4 in. thick at the top, the inner face being vertical. The depth of the flume is 3 ft. 1 in., and the width at the bottom ranges from 5 ft. to 5 ft. 6 in., according to the grade, which varies from 0.14 to 0.10 per cent. The capacity is 65 sec.-ft. with a 6-in. freeboard.

Where pier supports were employed the length of the span between piers was 25 ft., and the flume walls were made 7 in. thick. The wall reinforcing in these spans consisted of ⅜-in. vertical rods, spaced 16 in. on centers and about 1 in. in from the inner and outer faces. These vertical rods were continuous through walls and bottom, being bent into proper shape from 12 ft. 6-in. lengths. The longitudinal rods, arranged as shown in the accompanying drawing, were placed in 36 ft. lengths, lapping at the points not less than 30 diameters.

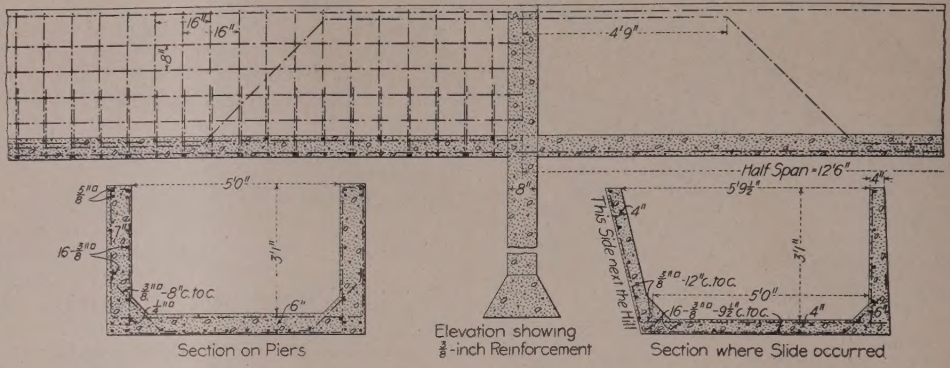
At a point where an old slide crosses the flume line it was decided to build a siphon 322 ft. long. A reinforced concrete pipe, 48 in. in

Flume Bridging Gap Washed Out by Overflowing Water

Concrete Pier Supports for Flume on Steep Sidehill

diameter, was adopted for this siphon, and was built with a 5-in. thickness, and reinforced according to the head, which was 40 ft. at the lowest point. A short section of the pipe was left uncovered at the lowest point and two 12-in. blow-off gate valves were placed here.

At the mouth of this siphon a short section of culvert type flume was used, as the ground was unstable at this point. This culvert section was called upon to resist a slide the first season after it was built. A large quantity of earth gravel and boulders was loosened from the overhanging bluff and swept down the steep hillside, passing over the culvert section without injury to the structure or interference with the flow.



Longitudinal and Transverse Sections of Congdon Flume

CONSTRUCTION FEATURES

Work was started on a 4500-ft. section of the flume late in January, 1912, and after the old wooden flume had been removed and the line regraded, a ½-yd. Smith mixer was brought on the job to be moved from point to point as the work advanced. For the most part it was found best to operate this mixer on the county road, which ran along below the flume, at points from which it was convenient to lay an inclined track up to the flume grade. A steam hoisting engine moved along with the outfit and supplied power for running the mixer and hoisting the cars up the incline.

The floor of the flume was poured first and after this had set sufficiently the wall forms were set, a runway was laid on the cross ties, and upon this concrete was conveyed along the flume for pouring the walls. Meantime graders, formsetters and steel men were strung out on the preceding sections.

The concrete mix used was approximately 1:2½:4. The sand was very clean and coarse; there was a natural, uniform gradation in the sand which ranged from about 30 mesh to grains the size of pea gravel. The gravel was passed through a 1½-in. screen. Both sand and gravel came from the river bed nearby and were screened and delivered by contract. The cement used the first season was Superior Portland, while Inland Portland was used in the second season. The concrete was mixed wet, well spaded so as to get as dense a product as possible, and finished with a two-coat neat cement wash. The steel was hard grade corrugated bars rolled by the Pacific Coast Steel Company.

The design of the flume was worked out taking into consideration the following stresses: (1) Contraction due to temperature changes; (2) water pressure, including a large factor of safety to cover the landslides and the impact of boulders rolled from above; (3) earth and frost pressure on the inner wall; (4) frost heaving from below, and (5) settling of the foundation.

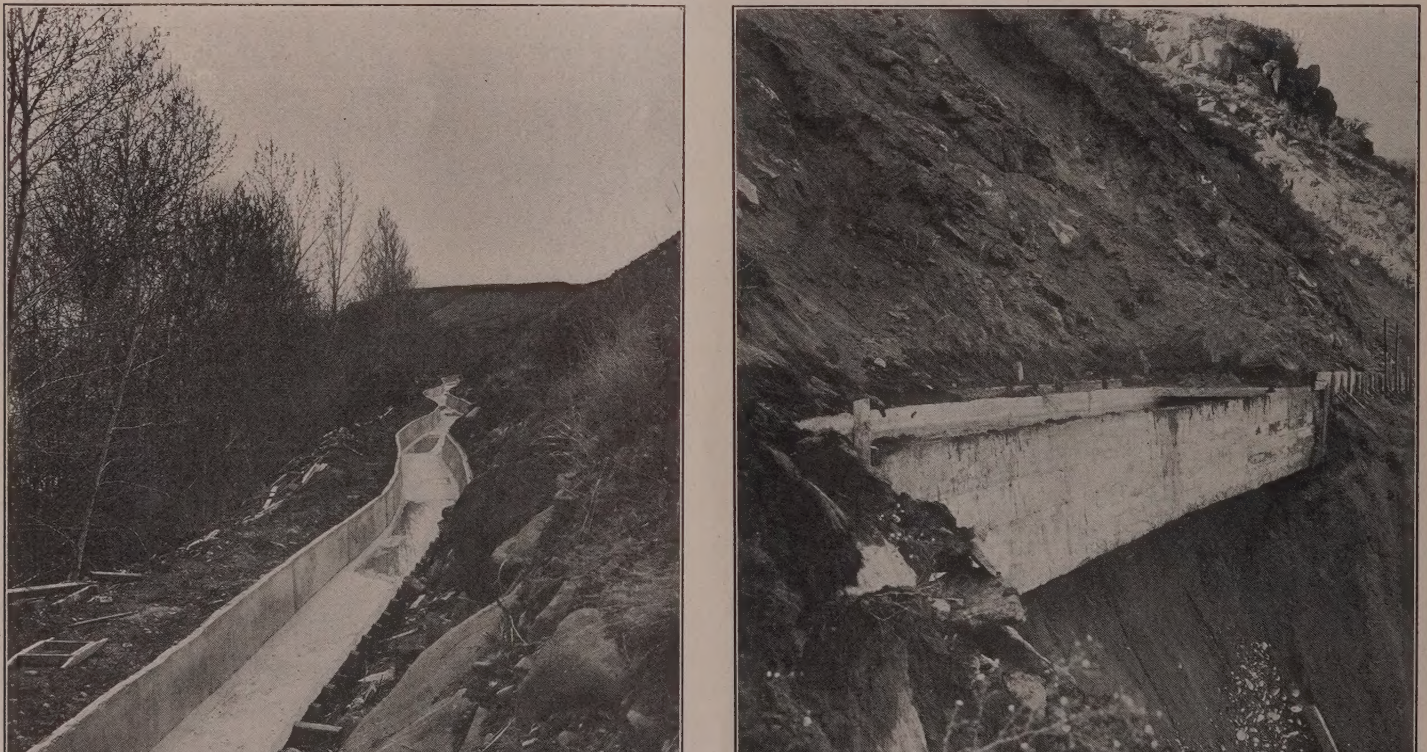
Considerable difficulty was encountered in placing the concrete on account of cold weather. All the work had to be done outside of the irrigation season, and in the winter of 1911 concrete was poured until Christmas. At that season the sun does not strike the flume at all, and on several occasions the temperature fell nearly to zero in the night. During freezing weather the water and gravel were heated as used, and after pouring, sheeting 90 in. wide was spread over the top of the flume. By the use of coal burning salamanders placed 50 ft. apart and kept burning night and day, as much as 1000 ft. of the work were protected at a time. After keeping the concrete warm in this manner for 48 hours it was allowed to freeze. In the following spring it thawed and resumed setting, and these sections are now said to be as strong as those poured in favorable weather. The only frost damage was a slight flaking at the top of the walls most exposed during pouring.

COST TABLE FOR 4500 FEET OF FLUME

	Total Cost	Cost per Foot
1140 bbl. cement at $2.50	$2,850	$0.633
1,070 yd. sand and gravel at $1.50 delivered	1.605	.357
Steel	918	.204
Coal	109	.024
Teaming	425	.094
Hardware, tools, etc.	342	.076
Form lumber	363	.080
Concrete labor	4,304	.968
Engineering and supervision	200	.044
	$11,116	$2.48
Grading	1,250	.278*
	$12,366	$2.758

*This does not include the expenditure of $1 per foot on the original wooden flume.

In the above table of costs, which refers to a portion of the flume founded on an excavated bench, the average cement and steel haul was 1 mile, and for about one-half the



Views of Flume before and after the Landslide which Caused Scouring Out of Foundations but Failed to Wreck Structure

work water had to be hauled to the concrete mixer, the average haul being half a mile. The lumber used in the forms and the labor of making them should not all be charged against this section of the work (as is shown in the table) because the same forms were afterward used on another part of the line. The cost of supervision is regarded as low because the superintendent divided his time between two jobs. The labor cost was 25 cents, and the teaming cost 50 cents per hour.

Flume Resists Landslides

In the summer of 1912 a slide started in the bluff above an open section of the flume and brought down a large quantity of earth and boulders of all sizes. This slide filled about 30 ft. of the flume, piling the material 3 to 5 ft. higher than the walls. The water backed up by the slide ran over the outer wall for several hundred feet, rapidly cutting away the earth from the steep slope beneath. The overflowing water cut the foundation from under the flume for about 50 ft. adjoining the slide, and although some material was left under the hill side of the flume, extending out beneath one-quarter of one-half of the floor width, this soil was left standing with an almost vertical face and had a very questionable bearing power. However, the structure stood intact under these conditions.

A still more severe test of the flume occurred on the night of Oct. 11, 1913, when a landslide dammed the flume near the mouth of the concrete siphon. Unlike former slides this one limited the overflow to the length of the flume between the point where the landslide filled it and the box culvert section. Thus the full flow of water was concentrated in a stream 30 ft. wide. This particular spot was known to be the most treacherous on the canal, and it was at first planned to carry the culvert type 200 ft. further along. This was not done, however, because the additional reinforcing bars specified for this type of construction were not on hand when needed, and there was urgent necessity for getting the flume in shape quickly to deliver water to the consumers. Thus there was left without special protection a short length of flume below which the steep slope consisted almost wholly of soft volcanic ash, and on this vulnerable point the recent landslide concentrated a flow of 40 sec.-ft., which continued for 8 hours before the supply could be shut off.

Bridging 41-Foot Gap

An examination made after the water shut off showed that a clear span of 41 ft. of the flume was left without any support whatever, the material having been cut away for this distance as far back as the inner corner of the flume bottom. At each end of the unsupported length the flume rested on soft earth the face of which had also been cut to an almost vertical slope by the overflowing water.

After clearing out the flume the full 40-sec.-ft. flow required by the consumers was immediately turned in again. Further examination of the unsupported section then showed slight moisture appearing at three places on the bottom of the 4-in. floor, but this moisture did not gather fast enough to drip, and the full flow was continued while three concrete piers were being built up to support the suspended section. When the piers are completed the flume will be decked over so as to continue the box culvert section beyond this slide zone, as was originally planned.

The flume was designed and the entire work of construction supervised by Mr. Edward A. Bannister, engineer for the Yakima Valley Canal Company.

South Yuba-Bear River Power Development

Description of Plants in One Valley in California and Progress Made on the First to Operate under 1375-Foot Head

By J. P. JOLLYMAN

Electrical Engineer, Pacific Gas & Electric Company

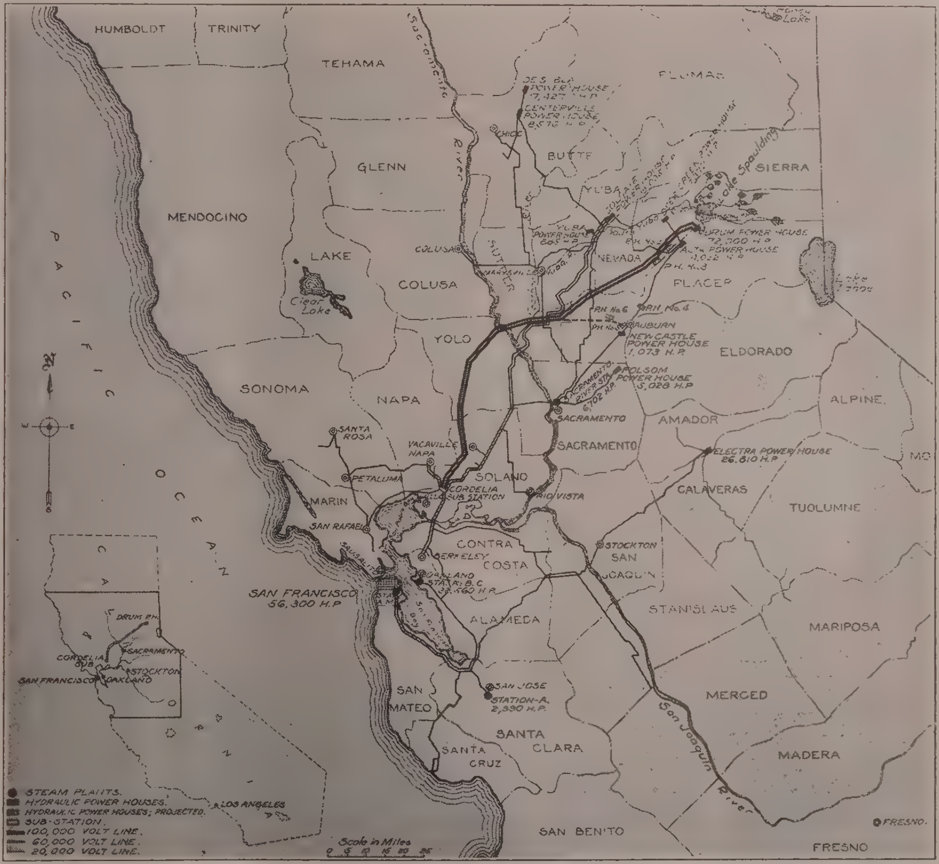
EARLY in 1912 the Pacific Gas & Electric Company, finding itself in need of more power to supply the rapidly increasing demand and of more water for sale in the irrigation districts of Placer County, decided to commence the further development of its South Yuba project. A careful study of the possibilities of the South Yuba project showed that the water from the upper watershed of the South Yuba River coud be used in seven hydroelectric plants, connected in series hydraulically, that the total of the static heads of these plants would be about 4100 ft. and that the total capacity of the plants would be 142,400 kva. After being discharged from the lower plants, the water would still be available for irrigating purposes in the fruit-raising districts of lower Placer County, where the company has been supplying water from its South Yuba system.

Of the several developments possible, half of the ultimate capacity of plant 1, or the Drum plant, will be installed and operating before the end of 1913. Considerable preliminary work has been done on plants 4 and 5 near Auburn, and it is expected that they will be completed during 1914.

The most striking features of this series of plants are the concrete dam at Lake Spaulding, described in the Engineering Record of Aug. 9, page 150, the utilization of over 4000 ft. head in the seven plants, and the ultimate use of the water for irrigation. The most valuable feature is that in the future more power can be developed by adding another unit in a plant already begun or another plant in the series.

Water Supply

The water supply for the project is obtained principally from the headwaters of the South Yuba River, whose natural flow is supplemented by flood waters stored in future reservoirs. The most important of these reservoirs will be Lake Spaulding, into which practically all other reservoirs will discharge. The canal system, which carries the water into the Bear River watershed and to the hydroelectric plants located along Bear River or near Auburn, starts at Lake Spaulding. The ultimate height of the Lake Spaulding dam will be 305 ft. and the reservoir capacity 4,000,000,000 cu. ft. The concrete in the dam is composed principally of cement and gravel. The gravel is obtained from a pit in Bear River near Colfax, loaded into bottom dump cars, transported over the Southern Pacific to the dam site and dumped into the mixer bins.

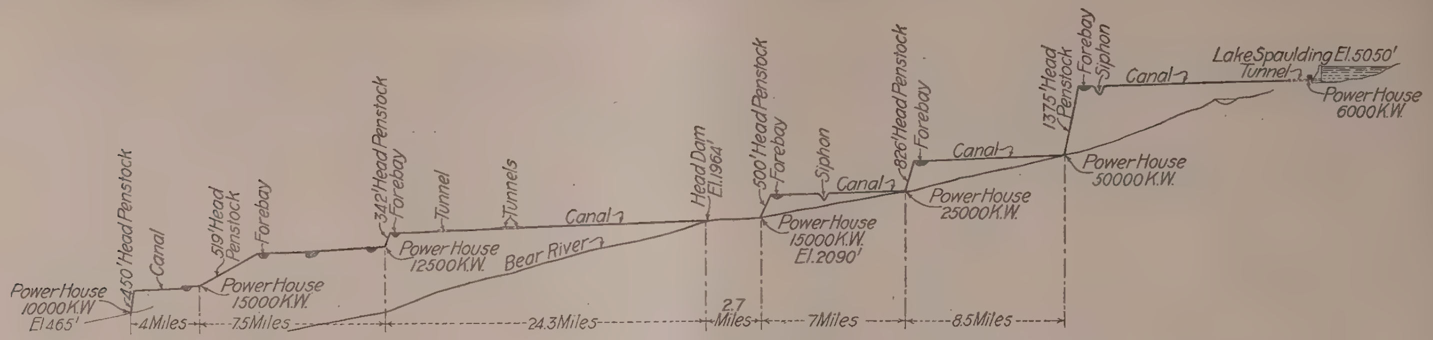


Plan of the Pacific Gas & Electric Company's Power Houses and Transmission Lines

During August, 1913, 40,485 cu. yd. of concrete were placed with an average force of 313 men. This makes an average of 4.2 cu. yd. per man per day and is believed to be a record for monthly yardage on this class of work as well as yards per day per man. The dam is expected to be completed to a height of 225 ft. before Thanksgiving, 1913.

The outlet is a tunnel in solid granite 4456 ft. long.

From the end of the outlet, a tunnel in solid



Profile Indicating the Relative Location of the Various Power Houses, Their Dams, Canals, Forebays and Penstocks

granite 4456 ft. long, a canal carries the water 8.4 miles to a forebay, which is an artificial reservoir of 17,424,000 cu. ft. capacity, created by the construction of earth embankments on three sides of a flat area on top of a hill. This forebay serves as a regulating reservoir for the power house, which is located in the Bear River canyon below.

The penstock will consist of two riveted steel pipes, tapering from 72 in. in diameter by ¼ in. thick at the forebay to 52 in. in diameter by 1¼ in. thick at the power house. Its length is 6214 ft. One of these pipes, with ample capacity for two main units, is now practically completed. The total head on the nozzles at the power house will be 1375 ft.

Power House

The power house is a steel frame building with reinforced concrete walls and roof, 208 ft. 8 in. long, 77 ft. 6 in. wide and 65 ft. high. It was practically complete on Oct. 1, 1913. Two 50-ton electric traveling cranes are provided in the main aisle.

Two 12,500 kva., 6600 volt, 3-phase, 60 cycle, 360 r.p.m. units are being installed and provision is made for the installation of two similar units. The shafts are horizontal and a 9000-hp. impulse wheel is overhung on each end of the shaft. Each wheel is driven by a single jet, controlled by a needle valve. Governing is effected by deflecting the jets.

The main transformers are 4250-kva., single-phase, and transform from 6600 to 125,000 volts. Several taps are provided on the low-tension winding to give line voltages ranging from 110,000 to 125,000. There will be one spare set of transformers. Low tension switches are so arranged that any desired combination of generators and transformers may be operated.

The 125,000-volt switching gallery is equipped with oil and disconnecting switches for the control of the two banks of main transformers and the transmission line. There will be two 400-kw, 125-volt, 514 r.p.m. exciters, each driven by a 600-hp impulse wheel and each equipped with a 600-hp induction motor which will serve to drive the exciter if anything should happen to the waterwheel.

The station is controlled from a switchboard of the bench board type, designed so that the entire station, with four main units and all auxiliaries, may be completely controlled both hydraulically and electrically from this one point.

By Oct. 1 both main units were completely assembled. The main transformers are being dried out in the power house, and the work on the switch-boards, switches and exciter is well advanced.

The energy developed will be transmitted to the Cordelia substation over a double-circuit steel tower line a distance of 109.5 miles. One circuit will be installed this year and the second circuit added next year.

From the power house the two circuits are carried on two lines of single circuit towers for 12.6 miles. These towers, which are 50 ft. high, support the wires in a horizontal plane. This section of the line was designed to avoid the possibility of trouble from the unequal loading of snow on the conductor. From the end of this section to Cordelia, where snow is practically unknown, the two circuits are supported on double circuit towers having one circuit on each side. The conduits here are placed in a vertical plane.

The double-circuit towers are 82 ft. high and spaced 800 ft. in flat country. There will be about 731 towers in the line and the average span will be about 790 ft. The conductors will be 3/0—7 strand copper or 5/0—19 strand aluminum. Copper is used from the power-house for about 31 miles or until the line is out of the Sierra Nevada Mountains, where the spans are somewhat irregular or snow loading may be expected. The aluminum is used for about 58 miles across the Sacramento Valley. Copper is again used for the last 21 miles, where the spans are somewhat irregular and sea fogs sometimes reach the line.

Cordelia Substation

The substation at Cordelia will be a reinforced concrete building 82 ft. wide, 256 ft. long and 47 ft. high.

The equipment will consist of two sets of three 4000-kva, single-phase transformers, transforming from 100,000 to 60,000 volts, and space will be provided for two additional sets and for a large amount of auxiliary equipment required to give service in the immediate vicinity of Cordelia.

Switching arrangements for 100,000 and 60,000 volts are provided for the incoming and outgoing lines, together with a switchboard for the complete control of the station.

Cordelia is a point from which a number of the existing 60,000-volt lines radiate. A study of the distribution of the loads on the system showed that Cordelia was very near the center of gravity of the load. These two facts, together with the convenience with which all points on or around San Francisco Bay could be reached from Cordelia led to the selection of this point for the main substation.

It will be completed to a length of 155 ft., and the equipment in this part of the building will be installed by the end of this month.

Mr. F. G. Drum is president, Mr. John A. Britton, vice-president and general manager, and Mr. A. F. Hockenbeamer, second vice-president and comptroller of the Pacific Gas & Electric Company. The construction work is under the supervision of Mr. F. G. Baum, as chief engineer, assisted by Mr. P. M. Downing, assistant chief engineer; Mr. H. C. Vensano, civil engineer, and the author, engineer of electrical construction. Mr. D. H. Duncanson is superintendent of construction of the Spaulding dam and Drum power house.

Penstock, Power House and Interior of Station of the Drum Development

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Forty Thousand Tons of Rock per Month for the last two months have been dumped from barges at the site of the government breakwater at Ogden Point near Victoria, B. C.

Overcoming Difficulties in New York Subway

Excavating Rock and Quicksand 50 Feet Below Tide Level under Traffic in City Streets — New Methods of Bracing and Street Support

Section 13 of the Lexington Avenue Subway extends 2914 ft. on Lexington Avenue from 117th to 129th Street. At 118th Street it is a four-track double-deck structure with steel beams and columns and concrete roof, walls and invert. North of this the cross-section is very irregular on account of cross-overs to sort trains at the double-level station at 125th Street, beyond which the tracks gradually run on to one level to pass through tubes under the Harlem River. The excavation varies from 34 to 75 ft. in width and from 37 to 60 ft. in depth.

Conditions

The street is built up solidly on both sides, chiefly with brick and stone buildings four or five stories high, the lowest stories being used for business purposes and the upper ones for residences. There are a few taller buildings, including seven-story apartment houses with steel or iron columns. There is a double-track street-railway line in the center of the street and a heavy traffic of ordinary vehicles during business hours.

The surface of the rock which lies from 2 to 50 ft. below street level is very irregular, sloping sharply both transversely and longitudinally in many places, so that it is sometimes 30 ft. higher on one side of the excavation than on the opposite side. At the south end of the section it is decomposed and very soft for from 2 to 4 ft. below the top. It is harder from 124th Street north, but is everywhere uncertain. Generally there is a cover of fine sand containing a little clay. Large numbers of boulders up to 10 yd. lie near the surface of the rock.

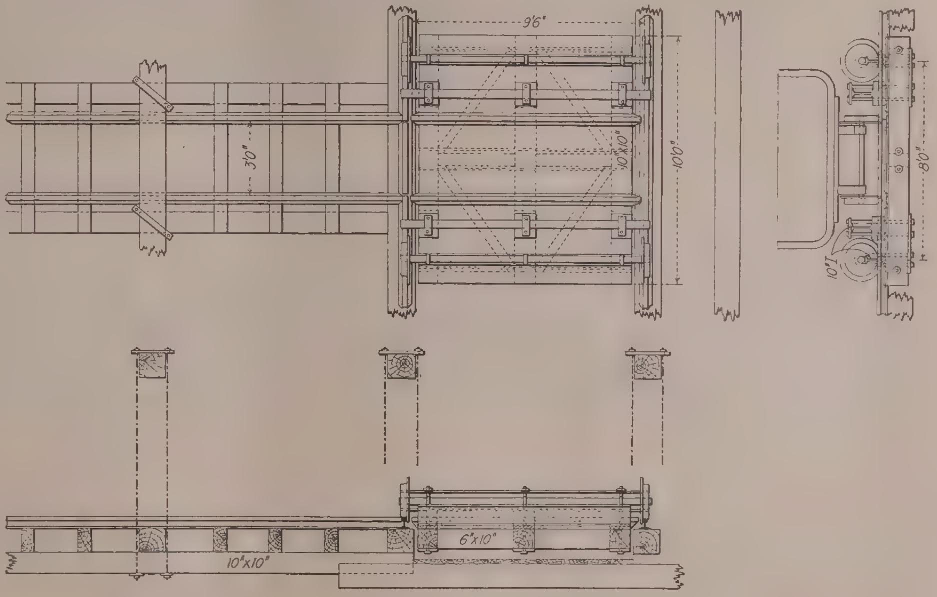
In some places large pockets of rotten rock are found entirely inclosed in sound rock. Quantities of ground water are encountered from 8 to 15 ft. below street level, making the sand there so wet that considerable difficulty is experienced in retaining it, and very heavy pressures are developed which, when the surface of the rock slopes transversely, are especially troublesome and produce unbalanced thrusts on opposite sides of the trench. Rock on one side and dirt on the other make it especially difficult to place the necessary horizontal bracing.

The complicated construction, great depth and width of excavation, the irregular conditions and treacherous materials encountered, and the necessity for carrying on the work without obstructing the street or interfering with traffic make the work very difficult. As a result new types of bracing and street support, as well as interesting methods of excavation and of handling spoil, have been developed.

The magnitude of the work is indicated by the contract price of nearly $4,000,000 and the principal items involved, which include 200,000 yd. of rock excavation, 130,000 yd. of earth excavation, 9000 tons of structural steel and 60,000 yd. of concrete. The contract was awarded to the Bradley Contracting Company, which commenced work in March, 1912, and afterward assigned it to McMullen-Snare & Triest, Inc., who commenced work in May, 1912.

General Methods

As on most of the other sections of this subway, the excavation is made in trenches covered by a temporary full-width wooden floor forming the street and sidewalk. Spoil is removed and materials are introduced by overhead apparatus at shafts located at intersecting streets. The shafts are provided with hoisting apparatus on one or both sides of the avenue. From them full-width headings are driven in both directions and the excavation is made with a 16-ft. upper lift and a lower lift to grade. Compressed air from a central plant is used for nearly all power purposes, hoisting engines being liberally used in the trench for handling all materials. Special pains have been taken to eliminate obstructions, so as to provide clear working space for the rock excavation, permitting the rapid delivery of muck to the dump pile without rehandling after it is loaded at headings, so that subsequent operations may be executed expeditiously and economically.



Suspended Track and Transfer Car for Handling Spoil Cars at Shaft

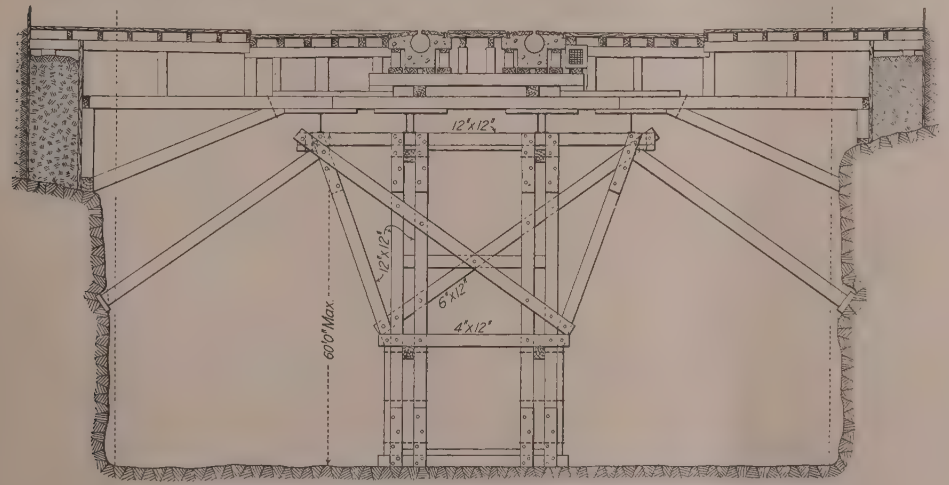
Sequence of Operations

The contractors have opened one shaft at 118th Street, two at 120th Street, one at 121st Street, one at 123d Street, two at 124th Street, one at 126th Street and one at 128th Street. Of the foregoing all are working except the one at 128th Street.

The original contractors excavated about 5000 yd. of earth, supporting the street by ordinary needles, shoring and cribbing. The gas supply was diverted in the usual way to surface by-passes alongside the curb, and the other pipes, water mains and conduits encountered here were supported on the timber work.

As the work is now being carried on, the first operation is the removal of the pavement for the full width in sections 20, 30 or 40 ft. long and its replacement by the wooden deck, carried on longitudinal timbers. Shafts 10 x 10 ft. or 15 x 20 ft. in plan, according to the hoisting apparatus installed, are sunk close to the curbs of intersecting streets, on one or both sides of Lexington Avenue, and are sheeted down to the surface of the rock. From them excavations 16 ft. deep are drifted transversely across the street, and excavation at this level is extended in both directions under the decking.

Underpinning of the adjacent buildings is carried on simultaneously with this excavation, and temporary connections are provided to maintain the operation of intersecting transverse sewers which are encountered. Most of the other substructures run into are comparatively near the surface of the street and are supported on the timbering to clear the permanent construction. The deck timbers and street-car tracks are supported on horizontal 12 x 12-in. transverse needles 10 ft. apart on centers; the latter are blocked up



Typical Street Support and Bracing in Rock and Earth Excavation

Falsework in Trench Supporting Girders Carrying Temporary Street Deck

and carried on a second set of full-length needles which have their center sections made of steel I-beams. These last-mentioned needles also serve as braces to receive the horizontal thrusts from the waling pieces that support the sheeting. They are carried on four lines of longitudinal 20-in. I-beams, a pair under each street-car track and a single beam under each side of street. These are received in 40-ft. lengths and are spliced together with bolted top-and-bottom-flange cover plates about 12 ft. long, making continuous beam supports.

Excavation and Girder System

The upper lift is carried through from shaft to shaft and is followed by the lower lift extending to subgrade and driven in both directions to meet between shafts. As fast as the second lift is excavated the longitudinal I-beams carrying the upper needles are supported on 11 x 14-ft. timber towers 40 ft. apart on centers, thus providing abundant unobstructed space for excavation, eliminating a large amount of timber that would be required by full-height posts throughout the work, and preventing danger to supports from blasting.

Adjacent to the face in working the lower lift, the 20-in. beams are supported on a transverse 30-in. Bethlehem beam 50 ft. long, spanning the heading which is driven on one side of the excavation. These beams are put up ahead every 30 ft. When the heading is sufficiently advanced under these beams, a massive block tower made of heavy timber, thoroughly braced and faced with steel to withstand blasting shocks, is placed under the end of the Bethlehem beam in the center of the street, and a second 30-in. beam is then placed in line with the first beam, but on the other side of the cut, with one end on the block tower and the other end on cribbing on solid rock outside the neat line. This second beam then carries the whole street and the first beam is taken down and moved ahead. The other half of the trench is then excavated.

When the excavation for the full width is completed the 30-in. beam supports are replaced by the 11 x 14-in. towers under the center of the street. All the towers and heavy timber work are framed in the contractor's yard and the completed bents are lowered into the trench and there handled and placed by the hoisting engines.

The system of continuous longitudinal beams is considered very convenient and advantageous and affords an opportunity for the installation of a system of diagonal transverse bracing reaching from the beam framework to the various tiers of waling pieces necessary for the heavy sheeting that retains the quicksand on either or both sides of the trench. This affords resistance for the heavy lateral pressure without the necessity of very long horizonal struts or the obstruction of the trench, and reduces the danger from blasting.

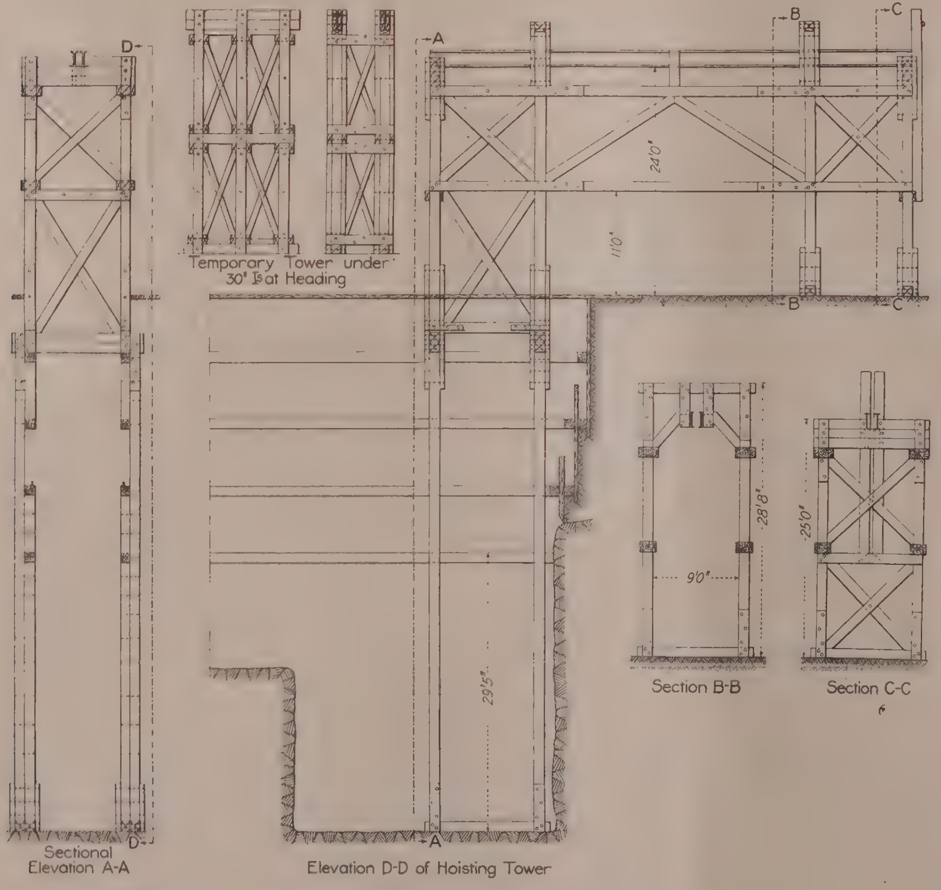
At 126th Street, on account of the many sets of horizontal bracing needed, the 30-in. 50-ft. I-beams are placed longitudinally, carrying the 20-in. I-beams and the street above over the face. One end of the 30-in. I-beam rests on a tower on the rock excavated to grade and the other end on the rock bench of the upper lift. This allows a clear working space approximately 40 ft. long and 60 ft. wide. When it becomes necessary to move the 30-in. I-beams the loads from the 20-in. longitudinal beams are carried down to subgrade by posts through the horizontal bracing.

Where the trench excavation is less than 40 ft. wide in rock the 20-in. I-beams are carried on transverse 30-in. Bethlehem beams 30 ft. apart; both ends of the latter resting on rock outside of the neat lines, thus allowing clear working space with no posts. Horseheads to grade are put under ends of the 30-in. I-beams close to side rock where necessary.

Method of Handling Rock at Face

The rock is drilled by about forty Ingersoll-Rand and McKiernan-Terry machines, driven by compressed air. These machines are operated by two shifts of drillers, one from 6 a. m. to 2.30 p. m. and another from 2.30 p. m. to 11 p. m.

The rock-excavation gangs and the main force start work at 8 a. m. and work until 5



Hoisting Tower at Shaft and Movable Tower to Support Girders in Trench

p. m. The drilling in the face is continuous during the time the drilling shifts are on, the mucking being carried on simultaneously from 8 a. m. until 5 p. m.

Blasting operations are confined as much as possible to the intervals between 7 and 8 a. m., 12 to 1 p. m. and 5 to 6 p. m., thus causing the muckers the minimum of delay during their hours of working.

A small gang of muckers is used in each heading from 5 p. m. to 1.30 a. m. to clean up difficult muck and to clear the benches for the drillers. These men also load a number of skips to be ready for the teams at 8 a. m. Practically all the heavy timber work and the moving of the towers and the I-beams are done by a force of men who come on at 5 p. m. and work until 1.30 a. m.

Steel skips of 84 and 104-cu. ft. capacity on 3-ft.-gage cars are used to handle both rock and earth muck. These cars as soon as loaded are pushed to the shafts, hoisted to the street, loaded directly on wagons and taken to the East 125th Street pier, where they are unloaded by two stiffleg derricks and dumped on scows moored alongside. The skips when loaded with rock weigh about 5 tons each. Very large pieces of broken rock are handled, being chained, dragged to position and placed in the skips by the hoisting engines in the trench. Hoisting lines are rigged through blocks hung from the I-beams carrying the street.

Shaft Hoists

Most of the shaft derricks originally installed have been replaced by trolley hoists of the telpher type, which occupy much less space, are more rapid and convenient, are decidedly cheaper and require only about one-half as much power for their operation. Each shaft is commanded by a pair of 24-in. I-beams 50 ft. long, 23 ft. in the clear above the street and having a clear span of 48 ft. The beams are spaced 5 in. apart and on their top flanges there travels a two-wheel trolley having two vertical steel plates 3½ ft. long projecting down between the track beams and carrying two hoist sheaves. The trolley is rigged with a tail line led around a fixed pulley at the end of the tracks and then returned to one drum of the hoisting engine. The line from the other drum of the hoisting engine is led around the sheaves at the opposite end of the trolley, thence down through the lower block of the hoisting tackle, up and around the other trolley sheave and back again to the lower block, where it is made fast. By this arrangement one of the lines pulls the hoist away from the engine and the other line raises and lowers the load and pulls the trolley toward the engine, thus securing a simple and rapid operation.

Transfer Platform

As the 126th Street shaft excavation is in progress on both the upper and lower lifts; the muck from the former is delivered to the shaft on service tracks laid on beams suspended from the upper needles so as to leave all of the space below them unobstructed for excavation in the lower lift. The full cars arrive at this shaft on a track, carried close to the east side of the excavation, and are delivered directly to the hoist. The empty cars are returned to the heading on a track near the center line of the trench and both tracks are connected by double cross-overs, which, together with 140 ft. of the east track, may be stored full of loaded cars, while the remainder of both east and center tracks is available for constant movement of full and empty cars. In order to provide for the movement of empty cars from the shaft to the center track a transfer car runs across the excavation at the level of the first lift, moving cars between the center track and the shaft in either direction. It is provided with rails parallel to Lexington Avenue, and engage either track. When the transfer car is at the shaft a heavy floor forms a protection over the lower lift; when moved toward the center track it leaves the shaft unobstructed for hoisting from the lower lift.

Personnel and Progress

The present force consists of about 150 men working at night and 550 men on the day shift. The work is being done under the direction of Mr. A. B. Lueder, superintendent, and Mr. W. J. R. Wilson, engineer for the contractor. Excavation is progressing at the rate of about 300 loads, or 600 yd., per day. On July 1 there were about 20 per cent of the rock and 35 per cent of the earth excavation completed, which, together with 70 per cent of the underpinning, made up 20 per cent of the total contract.

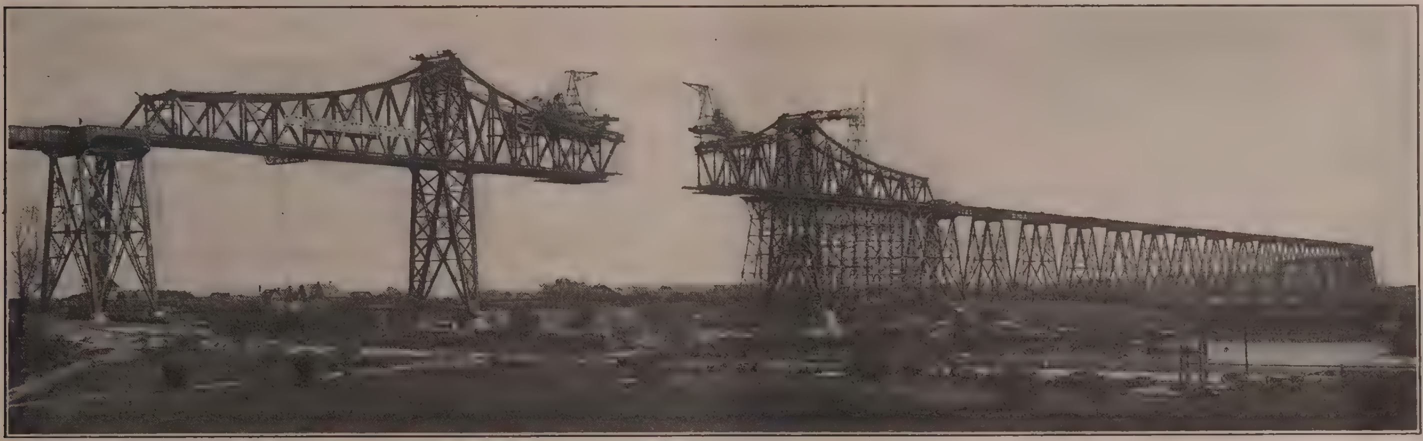
The work is under the direction of the Public Service Commission, of which Mr. Alfred Craven is chief engineer, Mr. Robert Ridgway, engineer of subway construction and Mr. C. V. V. Powers, division engineer.

State Road Aid is to be extended to villages and to cities of less than 5000 population in Wisconsin. The laws of 1913 provide that the main street directly connecting portions of road now on the county system of highways may be added to the county system of prospective State highways at the November meeting of the county boards.

Erecting the Bendsburg Cantilever Bridge

The bridge across the Kaiser Wilhelm Canal at Bendsburg, Germany, is about 8167 ft. long and weighs about 16,400 tons. The 459.2-ft. center cantilever span and the two 295.28-ft. side anchor spans weigh 3400 tons. The approaches have a combined length of about 7118 ft. and have towers from about 66 to 111 ft. in height connected by 93½-ft. plate girders. The towers supporting the bottom chords at the ends of the center spans are wedge shape in longitudinal elevation, with the apexes supported by pin bearings on the tops of low piers.

The approach towers were erected by a traveler operated by a Lidgerwood hoisting engine and the 50-ton girders connecting them were riveted up in the shop and erected by an overhead cantilever traveler. The anchor arms were erected by a gantry traveler running between the trusses, which were supported on wooden falsework. The cantilever arms were erected by small overhead travelers with a tower and cantilever arm mounted on plate-girder platforms adjustable to the varying angle and anchored to the inclined top chords of the trusses. The cantilever arm was equipped with a trolley hoist operated by a hoisting engine installed on the platform of the traveler.



Erection of the 459-Foot Span of the Bendsburg Bridge with Trolley Hoists on Jibs

Trolley-Wire Slack Distributor for Lift Bridge

In the design of the Cherry Street bascule bridge at Toledo, Ohio, over which the Toledo Railways & Light Company operates cars, no towers or overhead construction were permitted, and a special arrangement was required to distribute the trolley-wire slack. Fixed trolley poles are located at the end of the approach and at the outer end of the bascule leaf. At a suitable intermediate point are a pair of poles pivoted near the roadway level, extending 18 ft. above and 10 ft. below. The bottoms are weighted so that the poles tend to stand vertically whatever the position of the draw. They are stopped at a certain angle when the leaf is nearly erect, however, so that the tops of the poles always clear the bridge by several feet. The location of these poles and of the pins which fix the angle between poles and bridge are such that in any position of the draw the surplus trolley wire is divided about equally on the two sides of the pivoted supports, and is always kept clear of the track. The device has worked successfully since early spring, the bridge being opened and closed a number of times every day.

Evanston Filter Plant

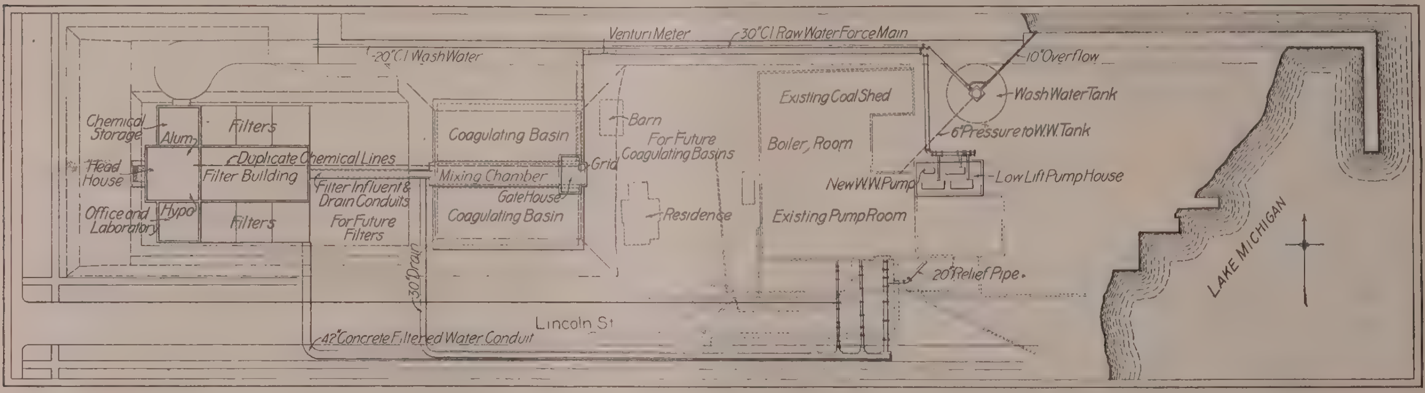
A 12,000,000-Gallon Rapid-Filter Plant with Ridge-Block Underdrainage System to Purify Water from Lake Michigan

Evanston, Ill., is building a 12,000,000-gal. rapid-filter plant to purify Lake Michigan water, which is now pumped raw to its 30,000 inhabitants after bleach treatment. The general layout was outlined in a report abstracted in the Engineering Record of Oct. 5, 1912, page 385, and the following notes are taken from a discussion presented by Mr. Langdon Pearse before the Western Society of Engineers.

The plant is located on Sheridan Road, about 1 mile north of the center of the city, on a site adjoining Northwestern University, 300 ft. from the existing pumping station. A low-lift pumping station is to be built adjoining the present pumping station. It will be equipped with three centrifugal pumps, two of 10,000,000 and one of 6,000,000 gal. per day capacity. These will be geared to steam turbines, the entire outfit being furnished by the DeLaval Steam Turbine Company under a guaranteed duty in excess of 69,000,000 ft.-lb. per 1000 lb. of dry steam for the larger units. A 30-in. cast-iron force main will conduct the raw water to the entrance of the mixing chamber, a Venturi meter being placed on the line. The mixing chamber is 96½ x 14 ft. in plan, 18 ft. deep and has a capacity of 170,000 gal., giving a nominal mixing period of twenty minutes. The chamber is divided into bays, 2 ft. wide, by wooden around-the-end baffles to maintain velocities of approximately 1 ft. per second. The mixing chamber lies between, and is part of, two coagulating basins, each 39½ x 96½ ft. in plan, 15 ft. deep. The basins have a combined capacity of approximately 830,000 gal., giving a nominal period of sedimentation of one hour and forty minutes. They are constructed entirely of reinforced concrete, have a groined-arch roof, and each compartment, to insure circulation, is divided by two concrete baffles into three bays.

Six Filter Units

Coagulated water will flow in a concrete channel across the space left for the extension of the filters to six rapid-filter units, each 23½ x 36 ft. inside, having an aggregate filter surface of 4425 sq. ft. The filters will be grouped on both sides of a pipe gallery, over which is a platform with an operating table for the hydraulic valves and gages for each filter. Effluent controllers are of a modified Vivian type. The wash-water drain, filtered-water conduit and raw-water conduit will be built one over the other, forming a reinforced-concrete structure down the center of the pipe gallery. The house, which extends over only part of the filters, will have brick side walls and a concrete roof. The operator can see the entire filter bed at the time of washing.



Relation of Tanks, Filters and Mechanical Equipment for Evanston's New Rapid Water-Purification Plant

Both filters and head house are placed above a covered filtered-water reservoir, 154 x 161 ft. in plan, 12 ft. deep, divided into two basins having a total capacity of 2,000,000 gal. This basin has a groined-arch roof, reinforced where necessary to carry the weight of the head house and filters. Arches are spaced on 12½-ft. centers and have a rise of 2 ft. 9 in. and a thickness of 6 in. at the crown. The foundation is of firm clay, so that the thickness of the floor arches is only 6 in. at the thinnest part.

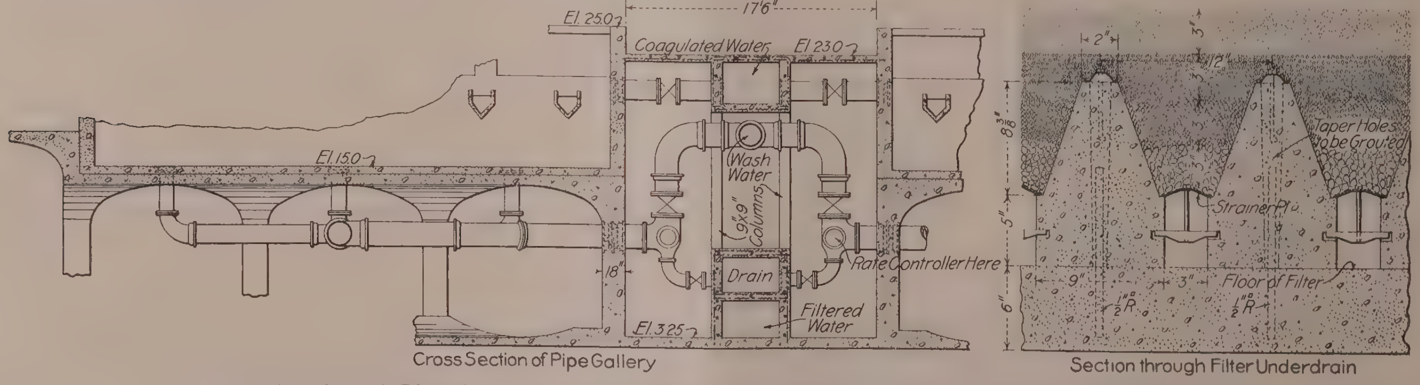
The two-story head house at the west end of the filter gallery has two wings, one story each, built of reinforced concrete and brick, with a tar and gravel roof. In the head house will be placed the tanks and apparatus for the proportioning and application of the coagulating and sterilizing chemicals, the recording gages, laboratory, office and chemical storage. The second floor of the head house will be served by a hydraulic elevator.

Underdrains

The design of the rapid-filter units includes a ridge-block underdrain system with perforated brass plates set at the bottom of valleys. A novel feature, however, will be the omission of the wire screen and the use of 12 in. of graded gravel, on top of which the sand will be placed. The wash is to be high-rate water wash. Filtered water will be supplied by a gravity feed from a 100,000-gal. steel tower, built by the Chicago Bridge & Iron Works, to be erected behind the present pumping station on the lake front. It will be fed by a bleeder from the present high-service mains. Fluctuations in the pressure due to the drop of level in the wash-water tank will be taken care of by a controller to be placed in the line at the entrance to the filter gallery.

Construction

Work is being carried on under a general contract with the Norwood Engineering Company, of Florence, Mass., the concrete work, head house and excavation being sublet to the Macdonald Engineering Company, of Chicago. Owing to the situation it was impossible to bring in material in any other way than by team, a haul of approximately 1 mile being required from the nearest railroad switch. The contractor, therefore, has developed a plant for elevating and storing the gravel and sand in bins to minimize the handling by hand. The teams dump into a hopper from which a flat belt conveyor raises the material to the sand or gravel bins. Storage sheds for cement are built alongside the bins. The Ransome concrete mixer discharges into two-wheeled buggies on an elevator for raising or lowering the concrete to the appropriate level for placing. One-inch yellow pine flooring has been

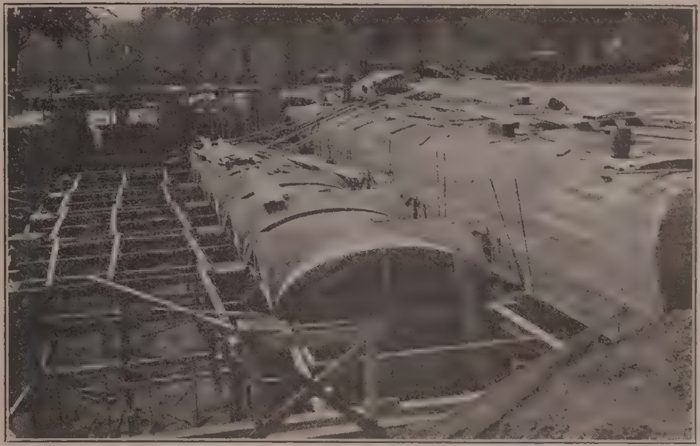


Cross Section of Pipe Gallery

Section through Filter Underdrain

Cross-Section through Pipe Gallery and Reservoir and Detail of Underdrains at the Evanston Filter Plant

Steel Screed for Inverted Groined Arches



Adjustment Space between Groined Arch Forms

used for formwork wherever possible, with 2 x 4-in. studs spaced about 16 in. on centers, tied with No. 18 wire.

The inverted groined-arch column bases were formed up continuously in rows by the use of a "spider," a metal frame consisting of 1-in. angles making a square, from each corner of which malleable castings of the shape of the groin converged to the base at the center, where they were held together by plates, as shown in one of the photographs. In use the angle frame was set on the two 6-in. floor form strips and held there by pins. After running a cut-off strip over each pair of legs and getting the base shaped, the form was removed to the adjoining base and the corners smoothed up.

Groined Arches

The groined-arch forms for the roof have been built in a manner different from that usually employed on such work. The form units are framed in the carpenter shop and transported to place on a pair of wheels. They cover the square area between adjacent column corners, leaving a width approximately that of the column to be filled on the job with short pieces of 1-in. flooring. This method was feasible because of the short span. The groined-arch forms were carried on stringers resting on the vertical tie studs of the column forms. The forms have been stripped at a minimum time of five to seven days, ten days' time being secured wherever possible. For the construction of the 30-in. wash-water drain and 42-in. filtered-water conduit suitable circular collapsible forms were built of wood. At the bends a built-up form has been used, made of laths, plastered smooth with Ivory wood-fiber plaster smoothed up with paraffine wax dissolved in kerosene. The forms were destroyed before removal.

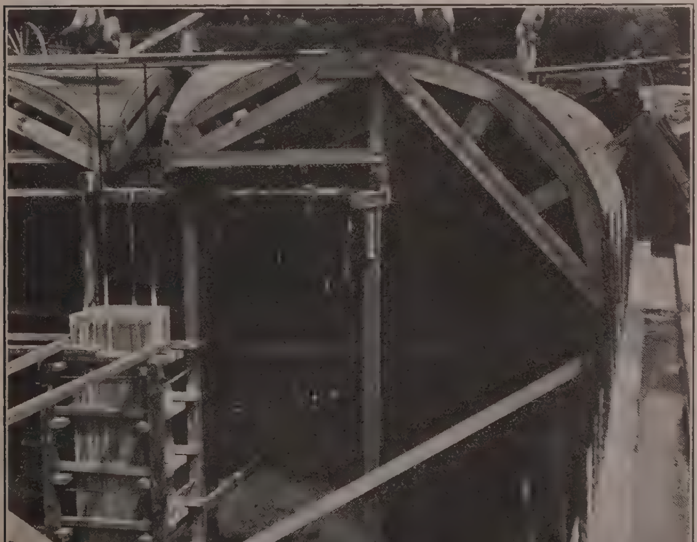
The excavation has been accomplished on all the mass work with a Thew shovel, loading two-horse bottom-dump wagons, with which the spoil was carried to adjoining lots or dumped on the lakefront. The earth was fairly stiff, so that no sheeting was required, except in trenches left open for days. The upper 5 ft. of excavation was loam and sand containing some water, under which a clay stratum was found free from water. Some of the sand from the excavation was suitable for use in the concrete, but the major portion of the concrete was made with torpedo sand and gravel brought from pits along the Fox River. The imported sand was found somewhat unsuitable owing to the lack of fine material, so that a mixture was made of 1 part plastering and 5 parts torpedo sand for general use. This made a very tight, nicely working concrete.

At present the work has progressed through the completion of the filtered-water basin. Two of the filters and the head house are in process of erection, and the coagulating basin is well under way. It is the intention to have the work under cover for the winter, so that it can be completed by May 31, 1914.

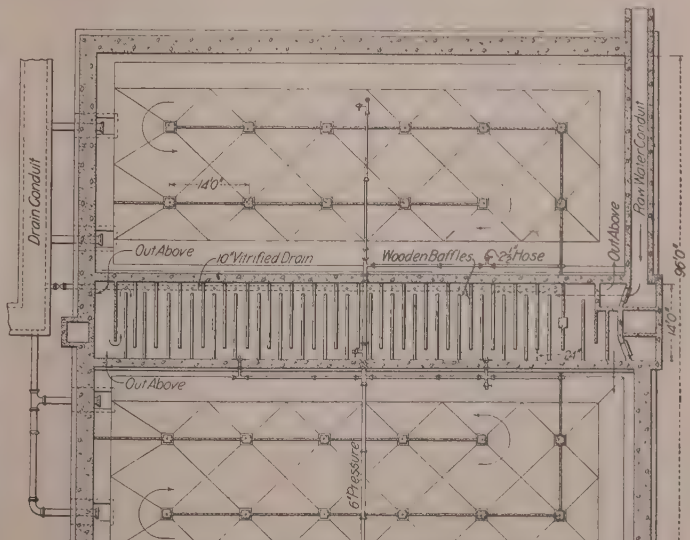
The engineers for the city of Evanston are Messrs. George W. Fuller and Langdon Pearse, of New York City and Chicago respectively. Mr. C. G. Gillespie is resident engineer. Mr. J. H. Moore is commissioner of public works and Mr. M. O. Kasson is superintendent for the contractors.

Stresses in Steel Reinforcement

The Prussian regulations for the design of reinforced-concrete structures were revised on April 22. The permissible unit stress of steel in compression and tension was raised from 14,000 to 17,000 lb. per square inch. The reason given for the increase is that the steel is rolled to such small dimensions as to increase its strength materially. The tensile strength for ⅜-in. bars should be at least 60,000 lb. per square inch and for 1 3/16-in. bars 54,000 lb. Intermediate values are derived by straight-line interpolation. With these values the elastic limit must be not less than 0.6 and not more than 0.7 of the ultimate strength. The elongation must be at least 25 per cent. For cold-bending tests the diameter on the inside of the bend shall be equal to half the thickness of the bar, and no cracks must develop. Proof that the material fulfils these requirements must be given to the authorities.



Forms for Groined and Barrel Arches



Plan of Coagulating Basins, Showing Baffles

Gravel Roads—Construction and Maintenance

Abstract of Paper Presented by S. Percy Hooker, State Superintendent of Highways, New Hampshire, at the Third American Road Congress, Detroit

THE economic value of gravel roads is such that they should constitute four-fifths of all the roads of a State. By "gravel" I do not necessarily mean a combination of sand and water-washed stone, but any combination of material which contains not less than 60 per cent of metal in shape and size so that it need not be crushed, whether the binder be true sand, clay or marl.

If the binder is of the latter, it must contain a larger percentage of metal than if composed of either sand or clay. To me gravel means an aggregate containing either a stone which from its own disintegration will form a binder or one to which must be added some material which is adhesive in wet weather and in drying forms a binding shell.

CONSTRUCTION

In most gravel pits there is stone too large to use in the surfacing but which serves well as a foundation either on sand or in wet clay holes. Telford is not absolutely essential under gravel; if it is used, the stones should be laid with some regularity. Stone as large as 3 in. may be used in the bottom course. The crown of the sub-grade should be 1 in. to the foot.

Surfacing may be accomplished by building the shoulders of other material and rolling the gravel 8 in. deep over the metalled surface of the road. I prefer to leave the rough grading with a crown of 3 in. to 10½ ft. (most of the roads we build are 21 ft. wide) and to give the gravel 10½ in. in the center and 3 in. on the outside edge of the road. This reduces the average thickness of the metal but gives a gravel shoulder which is invaluable in the maintenance. It also gives a 10-in. depth for the 5-ft. strip in the center upon which the major portion of the travel comes.

The gravel should be laid in two courses. By the first method this affords proper compactness; in the second it is not so imperative, provided the gravel is self-binding and the load is dumped far enough ahead so that it must be completely forked and shoveled over. Where laid in one course it is very easy to spread the gravel so that the larger stones are all in the bottom of the road, keeping ahead of the work in this way and leaving the surface composed of the finer material.

If a binder consisting of clay or marl is to be used two courses besides the binder should be laid. It is advisable to work the clay course thoroughly through the top surface with a harrow.

Frequently enough rain falls to allow the building of the road without the use of a sprinkler. This summer has been extremely dry, however, and for a considerable time after completion the roads upon which no water has been used have failed to "come together." I find, however, that a good soaking rain will compact a soft road readily, and where sprinkling is expensive it is wiser to build without it.

COST

The cost of gravel roads will vary to a greater degree than that of either bituminous or water-bound macadam. The pit price of the gravel may be only 5 cents per cubic yard; its cost is fixed by the average haul. With the material very near, the cost of gravel construction may not exceed $1,600 a mile, while with a 2-mile haul it may reach $3,500. Assuming grading and drainage expense to be $1,200 per mile, the total cost per mile varies from $2,800 to $4,800. The average of all the gravel roads built in New Hampshire has approximated $3,900. It must be remembered that many of these roads are in remote sections, where the cost of the more expensive road would be far higher than the average in States where the railroad facilities are better.

SPECIAL TREATMENT

Naturally with the coming of bituminous roads of the macadam class it was hoped that by the use of the same bituminous materials equally good results might be obtained on gravel roads. It was soon evident, however, that very little was saved by the attempt to use gravel in place of crushed stone in combination with bitumen. In order to make a satisfactory aggregate the gravel must be screened into several sizes and the natural binder must be eliminated.

The next step was to attempt the blanket treatment, using the bitumen on top of a gravel road, and many successful results have been recorded. I think one of the faults is that the untreated road, if constructed properly, contains the same native binder, which interferes with the construction of a bituminous road. I also think it impracticable to use as heavy an oil or tar as can be used on a macadam road, but the bitumen known generally as the 50 to 60 per cent class, applied hot and requiring a cover, frequently proves satisfactory.

Variations of surface treatment are made by the application of light oils and tars which can be applied cold and without further addition. In some instances these seem to be effective and satisfactory. The surface after application and under automobile traffic has in certain cases "rolled out" so that you have a fair imitation of an asphalt road, particularly in the wheel tracks. The tendency is, however, that where there is considerable horse traffic the hoof tends to "meal up" the road.

In most instances I think that the lighter oils on gravel roads only act as dust alleviators. The cost is cheaper than the other treatment, and where there are many homes along the road something must be done to render life livable. My general plan, however, places gravel roads only where residences are few.

MAINTENANCE

A year's experience has confirmed my statement at the 1912 Road Congress that, under the patrol system, a gravel road could be maintained in excellent condition at a cost of not more than 25 per cent of that of the higher types of roads. It has further strengthened my belief that the patrol system of itself is not enough to properly maintain the highways. I suggested that the ideal system of maintenance was by the use of a repair gang or a flying squadron, who should do the preliminary work in the spring necessary to put the road in good condition before it was turned over to the patrolman for his exclusive care.

All the roads which were handled early by this system are in better condition than when the patrolman started the care of them in May. The roads which were not in first-class condition when the patrolman began work have required extra repairing. In practice whenever the repair gang did not repair the road and wherever the patrolman was not able to show an improvement I have authorized him to procure assistance.

100-MILE UNITS

Starting with the "overhead," the division of the maintenance into units of about one hundred miles in charge of an inspector and the subdivision into patrol lengths of from 6 to 8 miles seems to me to offer the only practical system. I have not changed my mind as to the advisability of separating construction and maintenance, though I realize that the consensus of opinion is against it. It is easy to say that any faults of construction developing after the completion of the road should be taken care of by the same man who superintends the construction. Practically, however, the "faults of construction" as compared to the wear of the road are infrequent.

The special advantage in maintaining gravel roads consists in the adaptability of the road drag or hone to this work, the availability of the material for repair and the low cost of resurfacing. With material fairly available the cost of 4 or 5 in. of gravel upon the road will not exceed $500, and with a patrolman on the road such resurfacing will not be required for from eight to ten years after construction. The yearly resurfacing charge as distinct from patrol maintenance will only be $50 or $60 per mile, while the resurfacing of other types of roads will probably require at least ten times that amount. The patrol cost will not be more than $100 per mile nor the repair gang $50, so that the total maintenance charge over a series of ten years will not be more than $200 per mile.

I do not take myself seriously enough to suppose that a warning from me will change the attitude of anyone in regard to the building of highways in the respective States. It is, however, in my opinion the greatest menace to the good-roads movement to-day. With the average cost of improved roads $11,000 to $15,000 a mile, with an average charge for interest on the investment of not less than $600 per mile, with the maintenance estimated at $800 per mile, when the day of reckoning comes and the small taxpayer realizes the cost to him, how can you persuade him that economically he can afford to encourage the expenditure of such sums upon a limited mileage in each state?

FORTY LOCOMOTIVES CONVERTED TO BURN OIL were put in service this month on the 129-mile division of the Canadian Pacific Railroad between Vancouver and North Bend, B. C. The change has been made on passenger, freight and yard engines, and oil depots have been established at intervals along the division. The work of converting engines for oil consumption is now under way for the entire British Columbia division (510 miles, between Vancouver and Field), it is reported, and the new fuel will be in use at an early date. East of Field the use of coal will be continued. Crude petroleum from California is worth about $1 per 42-gal. barrel in Vancouver, and it is estimated that coal costs the railroad about $3 per ton. The use of oil fuel removes the necessity for maintaining a fire patrol along the railroad, which is required of coal-burning roads by provincial statute.

Concrete Pile Specifications

Method of Construction and Installation, Loading Tests, Settlement and Capacity

Three methods of making piles are permitted in the recently awarded contract for the piers and abutments of arch spans in the quarter-million-dollar approach for the Point Bridge at Pittsburgh. The estimated amount of piling totals 28,800 lin. ft., and the methods of manufacture provide for the use of a permanent tapered shell or a temporary cylindrical shell for casting in place, or individual molds for casting above ground. The piles were required to have a penetration of 5 ft. or more in the underlying gravel strata indicated by drill tests, and the manner of driving and casting was carefully specified. Provision was made for the application of a test load to one pile in every cluster and for the addition of extra piles, provided the test loads causing ⅜ in. settlement were less than double the nominal working loads.

Three alternate types, A, B and C, were specified. Type A is formed by driving a 16-in. steel pipe and inclosed mandrel, removing the mandrel, placing full-length reinforcement bar in the center of the pipe, filling with concrete and finally withdrawing the pipe. No heavy ramming sufficient to cause an enlargement of the hole at any point is permitted. All piles in one row must be driven before those in adjacent rows are driven. Adjacent piles must be driven before initial set or after final set of the concrete.

Type B piles have a permanent conical steel shell, 8 in. in diameter at the point and 18 in. in diameter 35 ft. above the point, where there is attached a pressed steel boot. The shell is driven with a tapered core, which is removed and the shell filled with concrete tamped around a vertical deformed reinforcement bar with 1-in. cross-section in the center of the pile. These piles may be driven in clusters with or without regard to rows, provided the shells are not filled with concrete until sufficient shells have been placed, so that the concreting is not less than 9 ft. distant from the shells being driven.

Type C piles of circular or octagonal cross-sections are premolded. Straight piles have a minimum diameter of 16 in. and tapered piles have a diameter of 9 or 10 in. at the points for octagonal and circular forms respectively and taper 1 in. in 10 ft. Each pile is fitted with a cast-iron or steel plate driving point and a thoroughly reinforced driving head, and the pile body is reinforced with longitudinal rods and satisfactory hooping or special winding to resist driving blows without injury.

TESTS

One pile in each group is selected for loading test, and if it fails to meet the requirements other piles in the group may be tested, and the capacity of the entire group per pile will be considered to be one-half of the test load, causing a settlement of ⅜ in. in the pile tested.

If this load is less than the load originally assumed additional piles must be driven to make their combined capacity adequate for the foundation loads.

A balanced platform is built on the top of the test pile and loaded with 30 tons, which shall remain undisturbed for thirty-six hours and then be increased to 60 tons and remain undisturbed for thirty-six hours more. Level readings are taken on a rod set on a steel dowel grouted into the pile. For each test nine readings are required: Before the platform is placed; immediately after the 30-ton load is placed; thirty-six hours after the second load is placed; when the 40-ton load is placed; when the 50-ton load is placed; when the 60-ton load is placed; thirty-six hours after the 60-ton load is placed; when the load is reduced to 30 tons, and immediately after the entire load and tripod have been removed.

To be acceptable the pile must not show a settlement exceeding ¼ in. between the first and third readings, or a settlement exceeding ⅜ in. between the first and seventh readings, or a variation between the first and ninth readings exceeding 5/16 in.

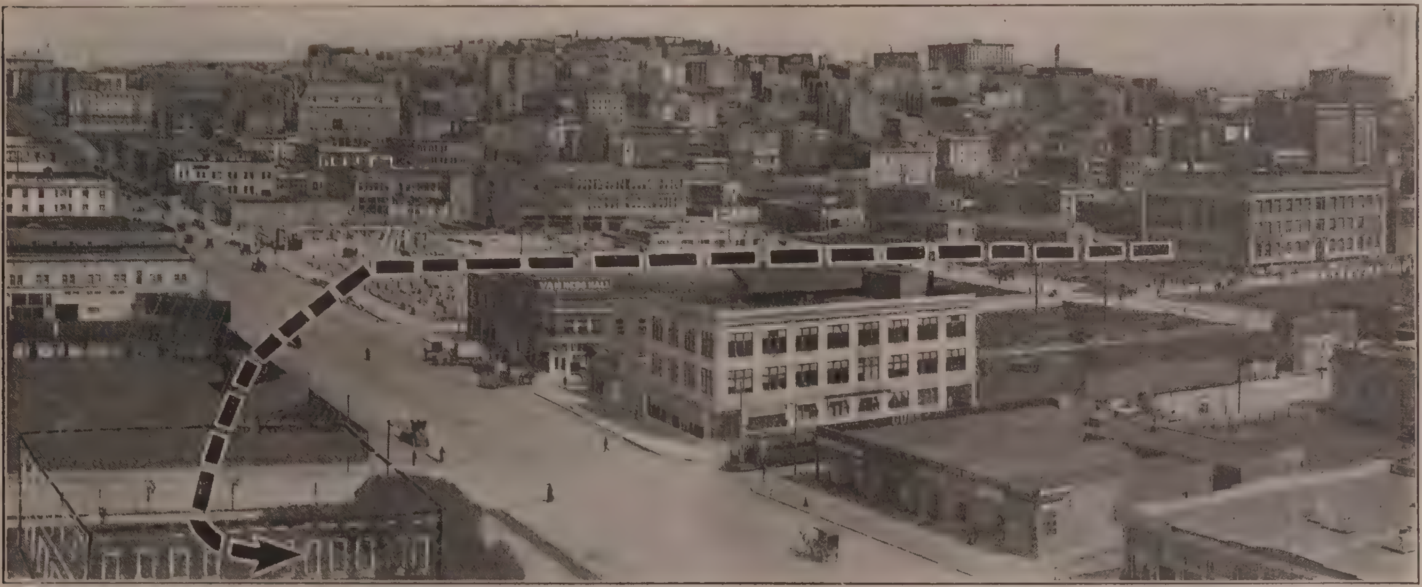
The specifications were issued by the Bureau of Construction, Department of Public Works, City of Pittsburgh, of which Mr. N. S. Sprague is superintendent.

AUTOMOBILE FEES IN MASSACHUSETTS in 1912 amounted to $492,482.50, according to the report of the State Highway Commission. Receipts from the registration of motor cycles totalled $9,644. Fees for operators, manufacturers and dealers, with sundries, brought the gross receipts to $616,236.44. All of this money was used for road maintenance.

Moving 8000-Ton Building

San Francisco School Transferred 1650 Feet at Rate of 40 Feet Daily

An 8000-ton three-story school building in San Francisco has been moved 1650 ft. from the proposed civic center site by placing it on a raft of timbers, cutting loose the supporting steel columns, hauling it across sand lots, four streets and two street-car tracks, and turning it through an angle of nearly 180 deg. The building rested on sixty 8, 10 and 12-in. Bethlehem-shape steel columns, the heaviest load on an interior column being 135 tons and on an exterior column 160 tons. It was located on the north side of Grove Street, between Polk and Larkin Streets, and was moved directly west to Van Ness Avenue, where it was turned somewhat less than 90 deg. about the center of the south wall as a pivot; then it was taken in a diagonal direction across Van Ness Avenue and Grove Street, blocked up over a cellar left by the fire of 1906 at the northwest corner of the intersection of Hayes Street and Van Ness Avenue, and moved across the two street car lines on Hayes Street to the northwest corner of the block at Franklin and Fell Streets, where it was again turned square so that the front faced south as in the original position.

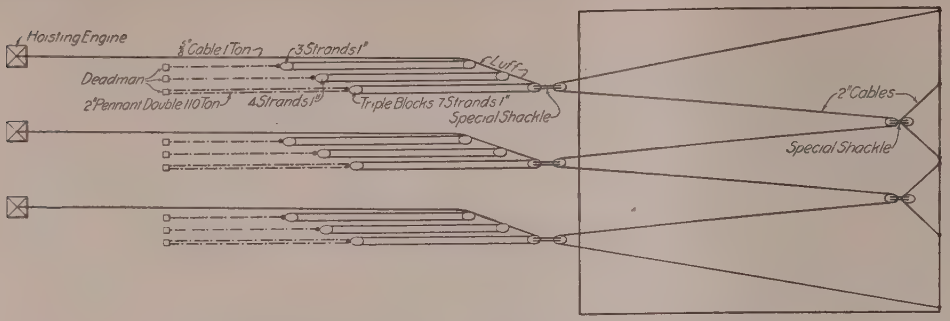


Route Taken by High School Building when Moved to Its New Site

In preparing the building for the journey it was shored up on 12 x 12-in. timbers running north and south under the floorbeams. The building is 142 ft. wide on the side facing Grove Street and 120 ft. deep. The columns are symmetrically spaced about the center of the 142-ft. side, but are quite irregular in the other direction, which accounts for the uneven spacing of the 14 x 14-in. needles under the 12 x 12-in. cross-timbers which were placed near the ends of each row of columns, even though there were only two columns in the row. Extra runs were placed near the south end to take extra-heavy wall loads.

SHORING RUNWAYS

As the footings for the steel columns extended below the ground surface into the basement the columns were cut off square about 4 ft. below the bottom of the floorbeams. Before removing the bases, templates for the splices were applied and holes drilled in the portion above the cut, while prick marks were made on each footing. As the interior load



Arrangement of Tackle and Engines for Hauling the Building

was carried not by the cut-off column but by supports near it, 12 x 12-in. shores were placed on four sides of each column from the first floor to the second. The illustrations show the details of the shoring runways, on which ½ x 6-in. steel plates were laid to receive the load from the 12 x 12-in. runners, 6 ft. long, under which were placed two similar steel plates turned up at the ends. There are sixteen of these runners on each of the nineteen runways and about 2000 of the 3-in. steel rollers 2 ft. long. More than 1,500,000 ft. of 12 x 12-in. timber were used in the raft and shoring. The blocking on the sand lot rested on 6 x 8-in. ties laid close together, with the material tamped under them as for a railroad. Above these a typical blocking, shown in the photograph, consisted of planed 4 x 10-in. timber lengthwise and 6 x 8-in. or 6 x 6-in. cross-ties shimmed up to the bottom of the two 12 x 12-in. runway timbers, over which the track plates were laid.

ARRANGEMENT OF HAULING CABLES

The hitch by which the building was pulled along can best be explained by the diagrammatic sketch. The pull was applied to six points at the rear of the building by 2-in. plow-steel Roebling wire cables passed around the 14 x 14-in. timbers of the raft. The two loops in the center permitted shackles to be attached at three points in the main cable. To these three shackles were attached triple blocks having seven strands of 1-in. wire rope leading 100 ft. away to the opposite blocks attached by a 2-in. pennant to the deadman buried as shown in the location sketch. From the triple blocks a luff was made to double blocks, also attached to the same deadman, and a second luff, with a single block giving three strands of the 1-in. rope ending in a ⅝-in. cable, led to the drums of the Mundy hoisting engines. The strain on the ⅝-in. cable was about 1 ton.

On the average the structure has been moved 40 ft. per day, but the time actually spent each day in moving was 30 minutes, the remainder being taken up in carrying forward the runways and rigging the tackle and blocks. As shown in the drawing, the hitch is such that any difference in the power of any engine is equalized in the main cable and simply results in bringing the triple block of its system together at a more rapid rate than the other two.

TURNING BUILDING

Turning the building at Van Ness Avenue was accomplished by placing steel I-beam supports under the raft approximately concentric around the pivotal point at the center of the north wall and placing the shoes under the arcs of these concentric circles. Hitches were then applied to opposite ends of the building to swing it around.

Originally the Newton J. Tharpe high school building cost $300,000. Bonds to the amount of $125,000 for the safe removal were given by the contractor, the Sound Construction & Engineering Company, of which Mr. James T. Walsh is engineer and manager. Nichols & Handley, as subcontractors, did the actual moving after the building was shored up.

Strength Tests of Bamboo

In many of the American island possessions in the East bamboo is the only material available for military structures. The trees often attain a height of 60 ft. and a diameter of 7 in. at the butt, where the fiber is sometimes nearly solid, though at the tip the thickness may not be more than ⅛ in.

Strength tests of this little-known material, made by Company H, Second Battalion of Engineers, were recorded by Capt. P. S. Bond in the September-October issue of "Professional Memoirs of the U. S. Army Engineers." Company H has built bamboo trestles of many types, with spans of 12 to 15 ft. and heights of 8 to 25 ft., capable of carrying heavy loads.

STRENGTH TESTS

Twenty small beams of bamboo were tested for transverse strength. The average fiber stress at the instant of failure was 17,000 lb. per square inch for seasoned bamboo and 16,000 lb. per square inch for green bamboo. The highest recorded test was 24,000 lb. and the lowest 10,000 lb. Most of the specimens were quite near the average. Rupture was always preceded by a considerable deflection.

BAMBOO BRIQUETTES

A number of specimens were tested for tensile strength by direct pull. The specimens were cut to a shape similar to a cement briquette, having a carefully measured minimum section. An astonishing strength was developed, as follows: Seasoned bamboo, average tensile strength, 29,000 lb. per square inch; green bamboo, average tensile strength, 23,000 lb. per square inch.

The highest recorded test was 34,000 lb. and the lowest 21,000 lb. The fiber near the outside of the tree was found to be somewhat stronger than that near the center. The minimum cross section of the pieces tested was ⅛ in. square.

The test pieces for the compressive test were ¼ to 3/16 in. square in cross-section and ½ in. high. The average strength in compression between two iron plates was as follows: Seasoned bamboo, 8800 lb. per square inch; green bamboo, 7000 lb. per square inch.

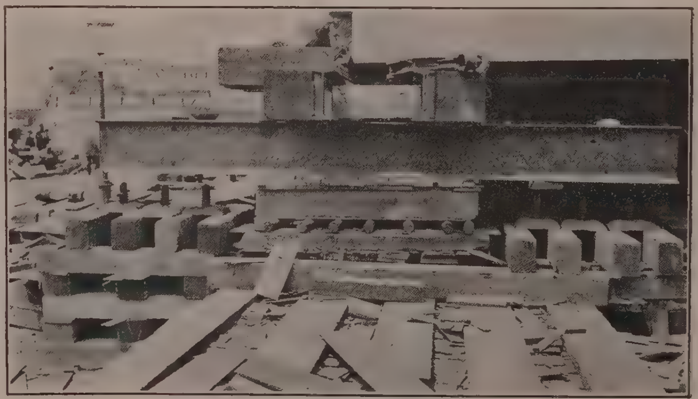
Failure resulted from splitting of the fiber as well as from direct compression. It is quite probable that specimens having a greater cross-section would have developed a higher compressive strength. It is to be noted that the transverse fiber stress was computed by the formula for beams, $P=My/I$, in which y was assumed as half the depth of the beam and M the bending moment of the known breaking load. The average depth, d, of the beams tested was 11/32 in. The difference between the fiber stress resulting from the application of the formula and the tensile stress as determined by actual tension is accounted for by the considerable variation between the tensile and compressive strengths of the material.

The following is the result of the tests of small columns. These had square ends and were tested between two iron plates. The first series of five were tested bare. The second series were wrapped at each end with marline. The first failed by crushing and splitting at the ends, the last by buckling near the center.

TESTS OF SMALL BAMBOO COLUMNS

Dimensions of column				
Length, in.	Diam., in.	Thickness of fiber, in.	Kind of bamboo	Breaking load, lb.
12	1	3/16	Seasoned	1,280
17¼	1 5/16	3/16	Seasoned	3,440
18	1½	3/16	Green	2,340
12½	1 1/32	⅛	Green	2,025
13⅛	1 1/32	⅛	Half seasoned	2,220
18	1½	3/16	Half seasoned	2,430
13½	1⅛	¼	Seasoned	3,800
13⅝	1⅛	3/16	Green	2,780
15	1¼	3/16	Green	3,480
15	1¼	3/16	Half seasoned	3,110
15	1¼	¼	Seasoned	2,360
17¼	1 7/16	3/16	Half seasoned	2,900

Steel-Shod Building-Moving Runners



I-Beam in Place for Pivoting Building

Elverum Hydroelectric Development

Plant, with an 11.8-Foot Diameter Reinforced-Concrete Pipe, Designed to Operate on a Norwegian River Where Frazil Conditions Prevail

THE Elverum hydroelectric plant, located about 5½ miles below the village of Elverum, Norway, utilizes the water from the Glommen River. The stream above this point, for 1¼ miles, has a fall of 36 ft. at low water and of 26 ft. at high water. The use of this site for power purposes had been discussed since 1890. On March 3, 1909, the village of Elverum appointed a commission to investigate the project and a franchise was granted soon afterward.

Two plans were advanced—one for the development of 1900 hp, at an estimated cost of $300,000, and another the development of 3800 hp, at a cost of $370,000. The city of Hamar bound itself by contract to utilize 500 hp during the first four years at a cost of $17.50 per horsepower delivered. After this period it was to be charged for an additional 300 hp at a unit price of $16.80. Power was to have been delivered on Jan. 1, 1910, or about twenty months after the contract was signed; it was not supplied, however, until eleven days later.

Hydrographic Conditions

The Glommen River, above the power site, has a drainage area of 5650 sq. miles. Fortunately, accurate river observations were available, as the Norwegian Government has maintained an observation station at the Elverum Bridge since 1871. The minimum discharge at this point is 1270 sec.-ft.; normally it is seldom below 1410 sec.-ft. During thirteen days in 1895, eleven days in 1900 and thirty-four days in 1903 the discharge had been below 1410 sec.-ft. The maximum observed discharge amounts to 88,300 sec.-ft., making the ratio between minimum and maximum 1:70. This ratio changes to 1:40 further downstream.

The upper end of the rapids of the Glommen River is divided into east and west branches—the Knap and Haugs rapids. Below them are the Skjefstad Rapids, with an available head varying from 24.2 ft. at 4600 sec.-ft. to 15.1 ft. at 70,600 sec.-ft. With larger quantities of flow this head is probably reduced to less than 13 ft.

The initial program called for the development of either the upper or the lower rapids, but it soon became evident that such a plan was excessive in cost as compared with the advantages of combining the rapids. The latter plan was therefore adopted. One of the most difficult details confronting the engineers was the prevention of ice troubles.

The upper end of the Skjefstad Rapids, where rock foundation was available, was the most desirable site for a dam. The dam, which is curved, is of the gravity-section type. It is connected at the west end to the wall for the intake canal. On account of ice and logs passing over the dam movable parts were avoided and the structure was designed as an overflow dam. The available crest length of the spillway is 1023 ft. and the discharge capacity is 105,950 sec.-ft.

The construction of the dam has resulted in the formation of a comparatively quiet lake. This enables surface ice to form at the intake and prevents further cooling of the water. During the winter of 1911-1912 severe frazil ice conditions prevailed all along the Glommen River, but at the Elverum plant no trouble was experienced. There are two sluiceways in the canal wall, one at about the middle, 13 x 5 ft. in area, and the other near the rack sill at the intake to the pipe line, 6.5 x 5 ft., in addition to which there is an ice chute.

Backwater Calculations

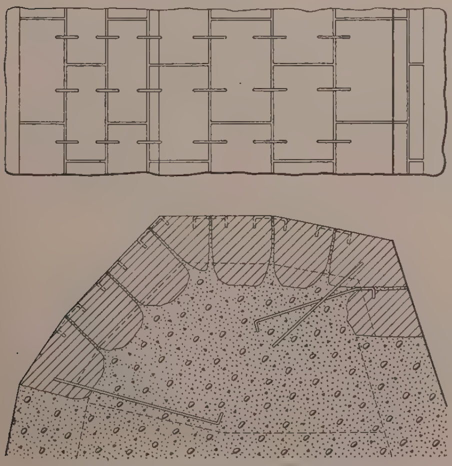
The following formula was used to determine the backwater from the dam:

$$y = \frac{v_1^2}{2g} \div \frac{v_2^2}{2g} + \left(\frac{Q}{c}\right)^2 \sum \frac{\Delta x P}{A^2}$$

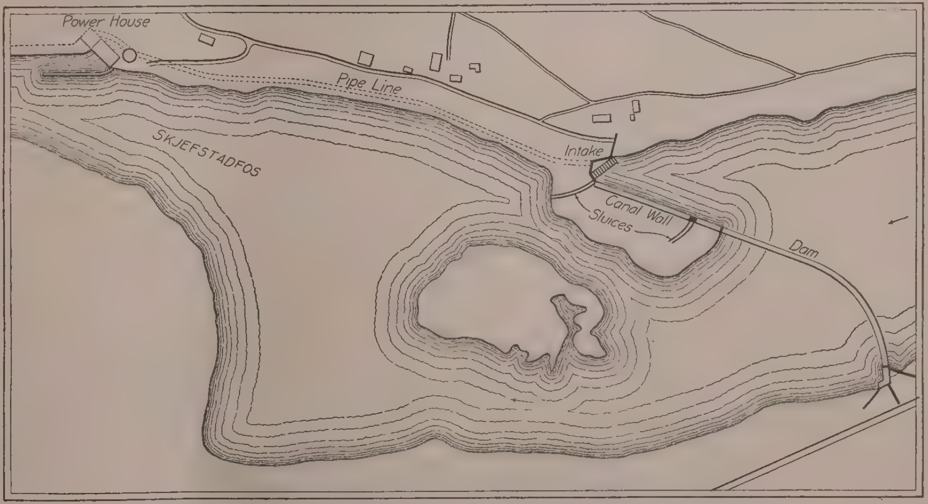
where v_1 and v_2 are the velocities at two succeeding sections; Q, discharge; c, coefficient; Δx, distance between the sections; P, mean perimeter; and A, mean area. The coefficient c was assumed to be 63 for the part of the river above the Knap and Haugs Rapids and 54 for the rapids themselves. It was calculated that at 4950 sec.-ft. discharge the slope of the water surface should be 2.43 ft. from the top of the rapids to the dam and at 88,300 sec.-ft. the surface slope should be 3.31 ft. During the flood of 1911, for a discharge of 47,000 sec.-ft., a fall of 2.72 ft. was observed and for 63,500 sec.-ft. there was observed a fall of 3.02 ft. The coefficient c, although given a lower value than would be expected for a river of this character, seems too high compared with the results obtained by means of the rating curve at Elverum. Later observations seem to prove that the adopted curve gives too small discharges, because the constant $\frac{2}{3}\mu$ should, if it were correct, be 0.35, which is entirely too low for a dam of this type.

As large quantities of sand and gravel of good quality were found near the site, concrete was used for the various structures. The dam and canal wall have their lower parts, sides and crests made of a mixture of 1 cement, 1.8 sand, 1.8 gravel and 1.8 broken stone, applied 2.5 ft. thick over a core of 1 cement, 3 sand, 3 gravel and 6 broken stone. Part of the back, the crest, and part of the front of the spillway are faced with granite blocks,

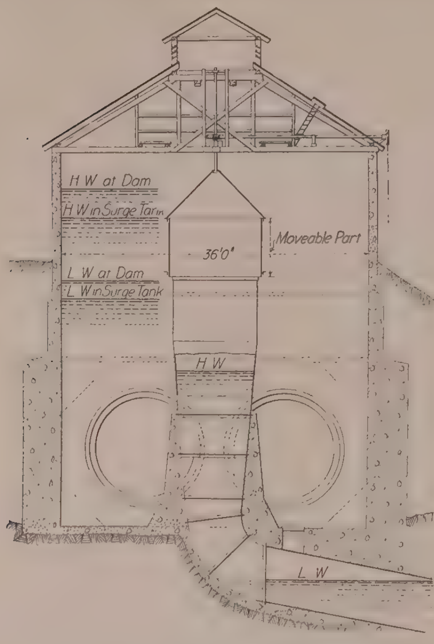
View of the Elverum Hydroelectric Development on Glommer River in Norway



Granite Facing on Top of Dam



Plan of Elverum Hydroelectric Development on Glommer River



Tank with Adjustable Overflow

anchored together and to the concrete by steel bars. Drains are provided at the lower upstream joint of the richer layer. There are expansion joints every 65 ft.

The canal, which is 675 ft. long, has a width of 80 ft. at the water surface and 60 ft. at the bottom. It occupies a natural depression and has a low-water depth of 12.5 ft. at the upper end and 19 ft. at the intake screens. While this seems unnecessarily deep, its construction was adopted to prevent clogging of the racks by ice slush. In the middle of the canal is a concrete sill, on the upstream side of which is a trench leading to the sluiceway in the wall. The sluiceway is operated by four cast-iron gates actuated by screw stems.

Intake

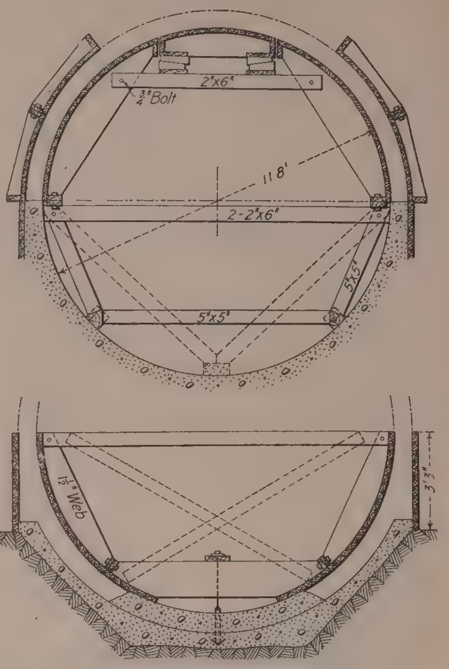
The intake at the end of the canal is protected by a screen, 78.5 ft. long, set at a skew so as to obtain a larger area and to facilitate cleaning. The bars are spaced 1 in. apart, and the sill is at El. 547.70. As the low-water level is at El. 563.50, the submerged depth of the screen is 15.8 ft. Back of the sill the bottom of the intake is depressed to El. 542.50. To prevent cooling of the water behind the screen the bars are covered with boards above the low-water level and the top of the intake is provided with a wooden floor. On the west side the intake is connected to the shore by a cutoff wall 25 ft. in length. This wall was originally built only 10 ft. long and was extended by a stone dike with a core 3 ft. thick, constructed of crusher screenings. The dike, however, was found to leak and the concrete wall had to be extended 15 ft. and carried down to bedrock.

The intake provides for two pipes, one of which has already been installed. They are bell-mouthed, from 16.4 ft. to 11.8 ft. in diameter. Each opening is closed by means of a steel plate-covered wooden gate actuated by a motor-driven rack-and-pinion device. Each opening has a wicket gate for relieving the pressure on the gate.

Pipe Line

The pipe rests on a concrete bed of lean mixture, most of which is on rock foundation. The diameter of the pipe is 11.8 ft.; its length is 1370 ft. The first 1168 ft. have a slope of 1:1000; but thereafter the slope is 1:24. The smaller slope was selected so as to place the pipe above low water in the river. It is curved in three places to a radius of 100 ft. As the quantity of flow varies between 740 and 950 sec.-ft., the prevailing velocity is from 7 to 9 ft.

In designing, two plans were investigated. One plan called for a steel pipe and the other for a reinforced-concrete pipe. It was then found that the latter was 25 per cent cheaper, when the cost of maintenance and its longer life were taken into consideration. Furthermore, the friction, it was believed, would be less in a well-made concrete pipe, and it was, therefore, adopted. As the Elverum district is one of the coldest in southern Norway, and as temperatures as low as 30 and 40 degs. Fahr. below zero are not unusual, it was found necessary, as a protection against freezing, to cover the pipe. During floods it is submerged for a length of 650 ft. The embankment, therefore, is plastered on the river side, and in one place, for a length of 300 ft. where the river velocity is too great, a dry masonry retaining wall had to be built. When the second tube is constructed it will be placed inside of this wall, provision having been made for its accommodation, but the plastered slope will have to be removed.



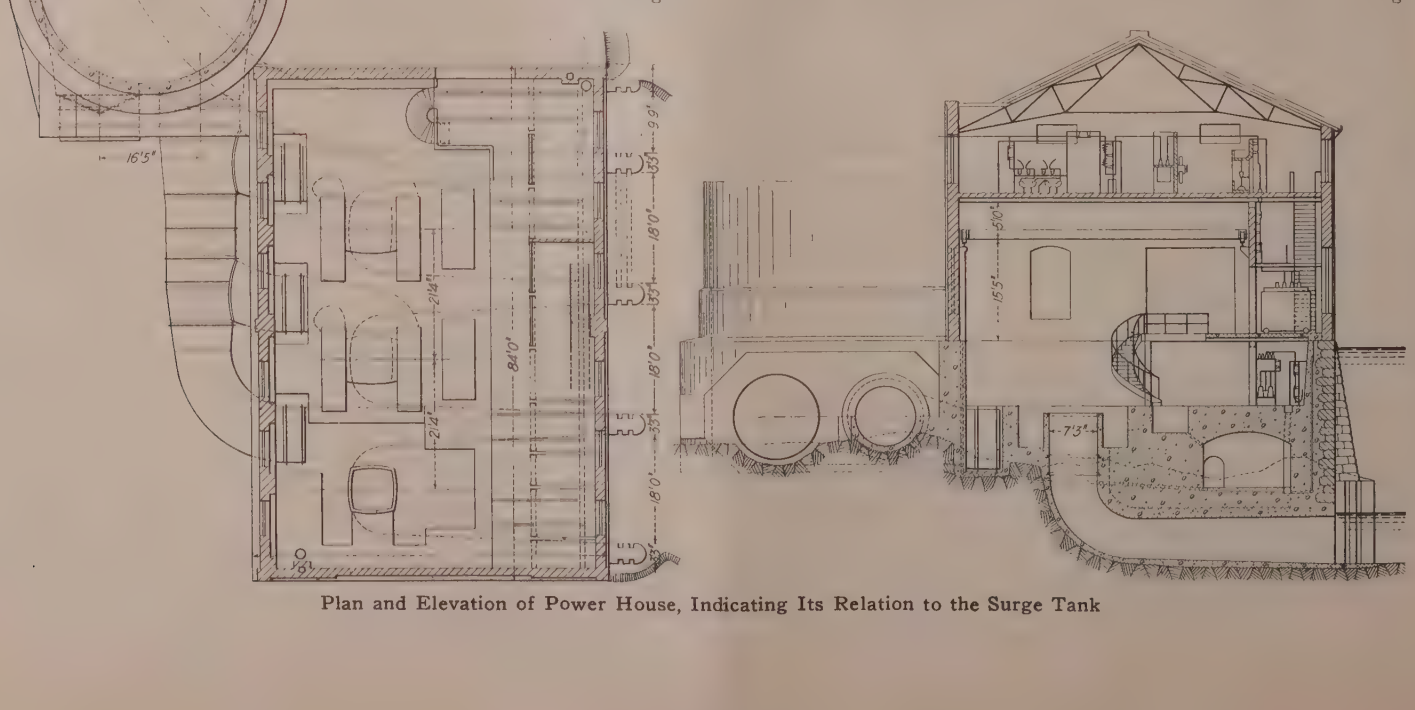
Arrangement of Forms for Pipe

No expansion joints are provided. In place of these the longitudinal reinforcement was made strong enough to take care of stresses due to changes in temperature. Steel rods ⅝ in. in diameter and spaced 4½ in. apart were used both transversely and longitudinally. The wall thickness is 8 in. Its inner surface was cleaned with steel brushes and then painted. Extra reinforcement was provided for at the joints required in the construction of the pipe.

As a protection against water hammer the concrete pipe terminates in a standpipe connected to the steel penstocks. This standpipe, which is 44 ft. high, has an interior diameter of 36 ft. In its middle is placed a steel overflow, the upper part of which is 10 ft. high and movable so that its crest can be adjusted to conform to the water level in the intake canal. The rising wave, due to water hammer, spills over into this pipe and discharges the water into the river.

Power Stations

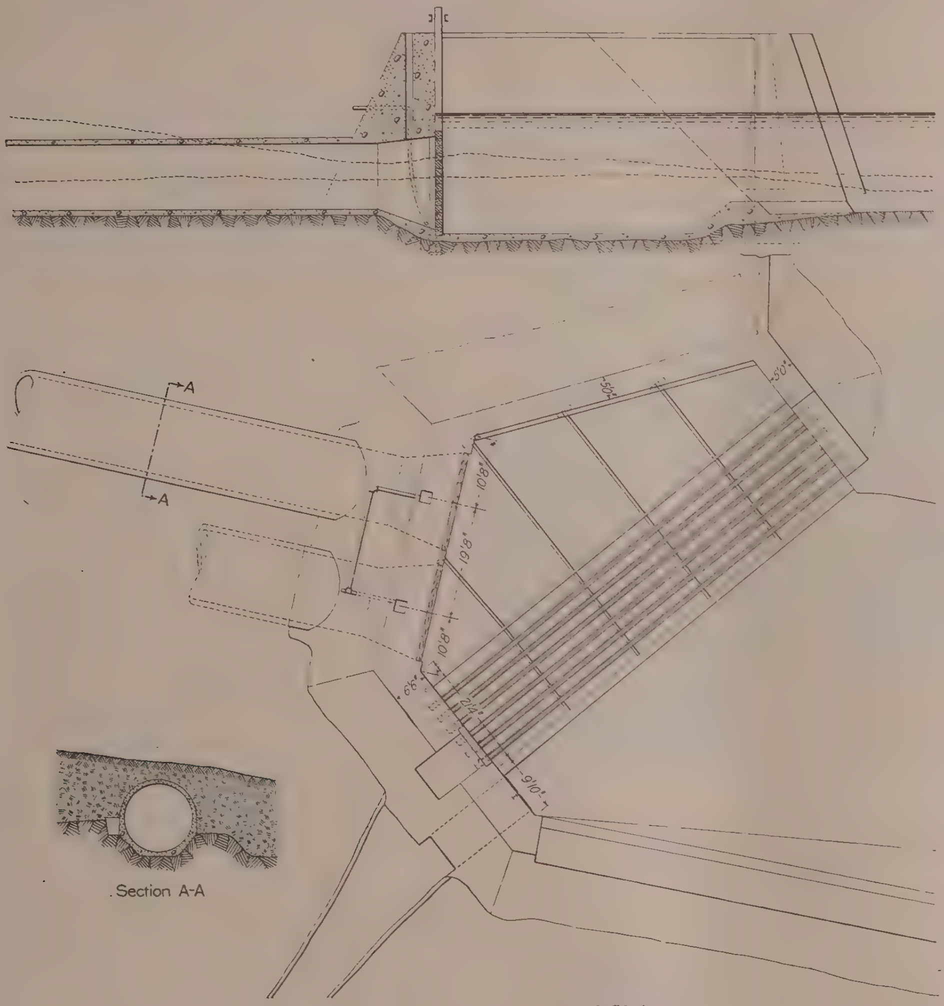
The present station is 84.6 ft. long and 59 ft. wide. When the new extensions have been built it will have five turbines of 1380 hp each, one of which will be a spare unit. At the present time only three units are installed. The water from the turbines is discharged



Plan and Elevation of Power House, Indicating Its Relation to the Surge Tank through concrete draft tubes into a tailrace 230 ft. long and 50 ft. wide. The lowest tailrace level is at El. 528.10 and the highest expected at El. 553.30. It was, therefore, necessary to waterproof the walls up to El. 554.30, as the floor is placed on El. 544.50.

The wheels are double-scroll case, single-discharge, horizontal Francis turbines running at a speed of 187.5 r.p.m. Under test they developed an efficiency of 87 per cent at full load, while the generators showed 94.5 per cent.

Direct-connected to the turbines are 1025 kva, 5000-5500-volt, fifty-cycle, three-phase generators. These units can be run in parallel or directly on the 5000-volt transmission line. The excitation current is delivered from 31-kw, 115-volt, direct-current machines placed on the generator shaft. One of these is sufficient to excitate both generators.



Intake for Pipe Line, with Screen and Sluiceways

Two three-phase transformers are installed in pockets at the downstream wall of the power house. They are 1050 kva each and step up the current from 5500 to 22,000 volts.

The plans for the hydraulic details of this development were prepared by the Kinck Hydraulic Construction Company of Christiania, and the plans for the electrical apparatus by Kittiksen & Steen. The contracts for the structures were awarded to the Hoyer-Ellefsen Company and for the steel structures and turbines to the Thunes Machinery Works. The electrical machinery and apparatus were delivered by Allgemeine Electricitäts Gesellschaft of Germany. The Kinck company also supervised the construction. Mr. Johan Kinck, owner of the Kinck Hydraulic Construction Company, supplied the information and the illustrations for this article.

Engineers as Municipal Department Heads

American Institute of Consulting Engineers Makes Plea for Appointment of Technical Men to Administrative Positions

The following letter, urging the appointment of experienced engineers as heads of some important departments of the City of New York, has been sent to Mayor-elect John Purroy Mitchel by the American Institute of Consulting Engineers:

The American Institute of Consulting Engineers is a national body of restricted membership, both as to number and qualifications. Membership therein is confined to professional engineers in independent private practice and of such prior experience and professional standing as enables them to serve their clients in an advisory, consulting or executive capacity, and the institute's membership consists, therefore, of only the older and more experienced of the engineering profession in its various branches, among whom are several of the past presidents of the American Society of Civil Engineers, American Institute of Electrical Engineers and American Society of Mechanical Engineers.

This institute has no candidates to present for office, and is, therefore, enabled to address you from an impartial standpoint removed from personal ambitions, in behalf of the best interests of this city and of good government.

We submit for your consideration what we believe to be strong arguments for the appointment of engineers as heads of these departments, and also reasons for believing that the economies in administration which you so much desire would result from carrying out our recommendations.

We recommend that the following offices should be filled by engineers: Commissioner of Bridges; Commissioner of Docks and Ferries; Commissioner of Water Supply, Gas and Electricity; Changes of Grade Commission (at least one member); Board of Water Supply (at least one member).

In thus urging the appointment of engineers to the heads of these departments we emphasize the fact that a selection should not be based upon technical attainments alone, and that high character, tact and business and administrative ability are absolutely necessary qualifications besides for anyone competent to fill these positions, and we believe that these departments cannot be administered with the efficiency and economy that you desire unless they are under the leadership of such types of engineers.

That these kinds of men are obtainable from our profession is proven by the fact that many private and public service corporations of this country have recognized this in selecting engineers for the highest executive offices. Presidents of the Pennsylvania, the Delaware & Hudson and many other railroads which are standards of excellence in their service to the public and are showing economical and financial results for their stockholders are engineers. Abroad we may take example from the cities of Germany, which largely owe their marked industrial, social and aesthetic advance to the engineering talent to whom they intrust the direction of their affairs.

An argument is often advanced by politicians and others that it is not necessary to have engineers as heads of departments, because competent engineers can be hired as subordinates. To this we answer that there are degrees of competency in the engineering profession, as there are in the legal and other professions, and that the higher type of engineer will rarely serve in a subordinate position in departments under laymen.

The same argument should be advanced as to the position of corporation counsel, or as to the head of the Board of Health. Why not place practical (so called) business men at the heads of these bureaus and let these employ legal talent or medical and sanitary experts? The answer is obvious. Experience has shown the necessity of putting at the head of these departments men who know their business.

It is noticeable that when members of the legal profession are appointed to office they are usually chosen from among the higher ranks. So in the selection of an engineer we ask that you consider only the engineer of experience and standing and with past performances to his credit.

The Commissioner of Bridges of this city is charged with the direction of the design, construction and maintenance of the bridges of the city, as well as the administration and operation of the Brooklyn Bridge and its transportation facilities, and under his jurisdiction there are about 4,000 employees. We cannot conceive of a private enterprise employing anyone but an engineer to direct an organization of this character.

The Dock Department has functions largely of a highly technical character, and a commissioner without practical engineering training and experience in the details of construction cannot but make mistakes costly to your administration and the public. On the other hand, to the competent engineer the execution of such details is the mere routine work of one familiar with the problems involved.

The Department of Water Supply, Gas and

Electricity is in the business of furnishing water, fuel and light to the city and the public—all matters of a highly technical nature, which in private corporations dealing with these same public utilities are in charge of engineers.

The Board of Water Supply being the administrative head and executive authority for the construction of this city's new water supply, has under its control the expenditure of more millions of money than the cost of the terminals of the Pennsylvania Railroad in this city. This latter corporation placed the sole responsibility of this great work in the hands of a commission composed only of engineers, while in the former there is not a single engineer on the commission.

The Panama Canal, with which the city's new water supply may be justly compared in point of cost and engineering importance, is approaching successful completion under the sole executive control of one engineer, thus justifying the policy of the Federal Government in this respect.

While the important details of construction of the new water supply is in the hands of a capable and experienced engineer, we submit that the interests of the city would best be served by the appointment of at least one engineer member to the Board of Water Supply.

The Change of Grade Commission we believe should also contain at least one engineer representative because of the technical and constructive matters with which it frequently deals.

Hence there are three departments and two commissions dealing wholly with the construction and maintenance of engineering works and the administration of municipally operated plants of a highly technical nature. These under any efficient business organization would be in charge of engineers trained by long experience in the respective branches of engineering to which their departments pertain, men who have given a lifetime of study to the problems involved and who have worked up from their apprenticeship through various positions to executive offices. Unfortunately, the past custom in this city has been to appoint to these offices men with no previous training or experience in the technique or affairs of the office they are called upon to direct. This results in errors very costly to the public and a loss of that efficiency and economy that you so justly seek.

We feel confident, therefore, that in urging upon you the advisability of appointing experienced and properly qualified engineers to direct the departments and to serve on the commissions mentioned we are acting in the best interests, not only of the city and of your administration, but of good government as well.

Overhead Bridge with Concrete Protection

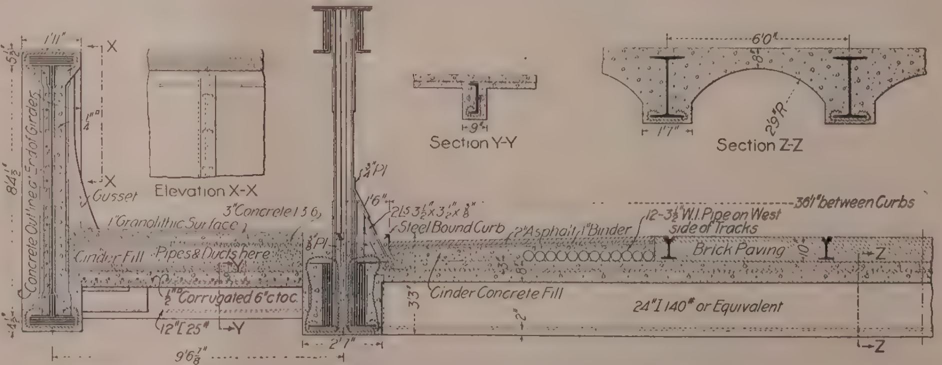
Inclosing Girders and Floor Systems to Insulate Them from Locomotive Gases

The single-span bridge carrying York Road over the Port Richmond branch of the Philadelphia & Reading Railway in Philadelphia has a low clearance over the tracks and is so much exposed to locomotive gas that a large portion of the steelwork has been entirely inclosed in a solid mass of concrete to protect it from the cutting and corroding influences of the blasts.

View of Span before Sidewalk Girders Were Inclosed with Concrete

The bridge, which is about 60 ft. wide and 127 ft. in maximum skew length, has two riveted trusses supporting the roadway floor of concrete arches on transverse I-beams and two outside plate girders carrying the concrete sidewalk slabs. The floor arch concrete is extended around the bottom flanges of the floorbeams and is reinforced and anchored by expanded metal and by horizontal bolts through the beam webs.

The bottom chords of the trusses are inclosed in solid masses of 1:2:4 concrete, continuous with the curb and having a rectangular cross-section about 2 ft. 7 in. wide and 3 ft. 3 in. deep. As these trusses are only little deeper than the outside girders and are more than 9½ ft. distant from them, it was believed that the latter would sufficiently protect them, and the trusses are not inclosed above the floor surface. The side girders, about 8½ ft. deep, are however, completely protected by concrete jackets, which not only cover all parts of the girders but are extended to form ribs integral with the web protection and covering the gusset plates and their flanges.



Solid Floor and Protection of Lower Chord of Main Truss

The bridge was designed by the engineering department of the Philadelphia & Reading Railroad, of which Mr. William Hunter is chief engineer and Mr. W. B. Riegner engineer of bridges. The superstructure was fabricated and erected by the McClintic-Marshall Construction Company.

Wood Block on Steep Grades

Special Pavement for Noiseless Driveway Entrance to New York Apartment House

The driveway pavement to the front and rear entrances of the Montana Apartments, fronting on Park Avenue between Fifty-second and Fifty-third Streets, New York City, are examples of the use of creosote wood blocks under unusual conditions. In the selection of the pavement for the approach to the building, which is of the high-class residential hotel type, the first consideration was noiselessness. Several other points, such as cleanliness, slipperiness and general appearance, had considerable weight in the final choice.

The driveway to the porte-cochère at the front of the building is on a level grade and closely resembles in shape a semicircle with its concave side toward Park Avenue, the alignment of the curb lines being compound curves. The peculiarity of this pavement is entirely the pattern in which the blocks were laid. To lay the blocks in straight lines parallel to the axis of Park Avenue had the disadvantage of an unpleasant appearance. Courses laid along lines radiating from the center of the circle were tried, but found to be unsatisfactory because they necessitated blocks slightly wider at one end than at the other. Otherwise the joints at the outer edge of the semicircle would have been too wide. After some experimenting a herringbone pattern was adopted for the portion of the driveway lying outside of the porte-cochère. No attempt was made to have the center line of the herringbone pattern follow the center line of the driveway, but, on the contrary, the center line or apices of the courses were laid in a general way perpendicular to the axis of Park Avenue. The width of each strip was made equal to the distance between the curbs. A border of three blocks in stretcher courses was placed on either side of the roadway.

Special Blocks.

The essential feature of the rear driveway was a special block and its use on a grade that has been generally understood to be prohibitive for creosote wood block pavement. The driveway extends along the rear of the building from Fifty-second to Fifty-third Street, a total length of 200 ft. Its width at the street intersections is 11 ft. but it broadens out near the middle of the distance into a 35 x 42-ft. rectangular area. As the rear entrance to the apartments is on the basement floor, there is a drop of 8 ft. from each end of the driveway toward the center. The resultant grades are about 10 per cent over a length of 80 ft.

The contractor at first was somewhat reluctant to lay wood blocks on so steep a grade. However, a noiseless pavement being absolutely required, granite block and brick pavements were eliminated without further consideration; asphalt had the disadvantage of being as slippery as wood blocks. Consequently it was decided to use a block 3½ x 4-in. in cross-section, with beveled corner. Before deciding on this type of pavement a study was made by the engineer of the action of the calks on a horse's shoe when hauling a load up a grade. Several observations were taken at a hillside street on Staten Island. It was soon noticed that the heel of a horse pulling a load uphill never touches the pavement, all the strain of the pulling action being thrown on the front calks. It, therefore, seemed desirable to use a block as narrow as possible and to retain the center of gravity at approximately the action line of the horizontal bearing area, so as to prevent the overturning moment throwing the block out of position. For this reason the blocks were made and laid with the 4-in. dimension vertical and the 3½-in. side upward. The beveled corner cuts off 1½ in. on the surface and 1 in. on the vertical sides. The blocks were, of course, laid with the beveled corner on the downhill side. The blocks were of yellow pine impregnated with a light oil having a specific gravity of 1.03 to 1.07; the minimum of oil was 20 lb. per cubic foot. The blocks were furnished by the Wyckoff Creosoting Company, of New York City, and the work was done by Mr. Richard Lamb, consulting and constructing engineer, of New York City. The plans were made by Mr. Lamb collaborating with Mr. William D. Saunders, engineer in charge of the Montana apartment building.

Dredging in Havana Harbor

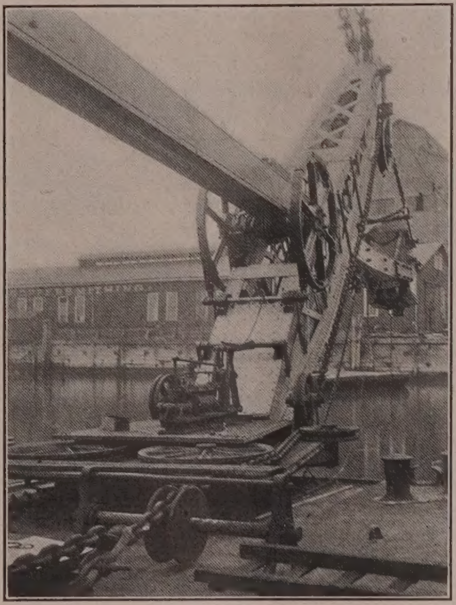
Improvements in the harbor of Havana, Cuba, include the dredging of 10,000,000 cu. yd. of mud to increase the depth of water from 24 to 41 ft. The work was commenced in 1911 and is to be finished in 1917. About 3,000,000 yd. have already been removed by the three dredges—Bismark operating a 10-yd. clamshell bucket, Craighill operating a 5-yd., and Sagua operating a 7-yd. clamshell bucket, all of them delivering to dump scows, which are towed to sea, where the material is deposited in water 6000 ft. deep.

The Sagua, of recent construction and improved equipment, has a 34 x 110-ft. steel hull 12 ft. deep, and is provided with a 53-ft. structural-steel boom weighing 26 tons and a 52-ft. structural-steel dipper handle operating a 5-yd. dipper to a depth of 35 ft. It is also equipped with a 7-yd. clamshell bucket of very simple construction, made with pressed-steel bowls, a structural-steel frame and a 48-in. closing wheel mounted in the center of the pivot shaft. It is provided with two black gum guide poles from Florida, which are 10 in. in diameter and 50 ft. long and maintain it in position in deep water.

The dipper handle has a full-length rack engaging a driving pinion connected to a sliding plate on top of the dipper handle and governed by a band brake operated by an auxiliary engine seated at the foot of the boom. The dipper handle is graduated in feet and inches, and the operator is able instantly to adjust it or lock it in position for work at any required depth.

The boom is mounted on an Osgood turntable and its guys are supplemented by a 2-in. steel slack preventer stay, attached to a 20-ton coiled spring to take up the impact in case the guys break. The boom is guyed to the top of an A-frame anchored by four 2-in. back stays, which, at the top, run freely over 24-in. sheaves designed to equalize their stresses.

The dredge is equipped with one double-cylinder 18 x 26-in. Flory main engine and two 10 x 12-in. engines to handle the spuds with tackles suspended from a steel gallows frame. The inside struts are 36 x 38-in. Oregon pine timbers 60 ft. long, and the stern strut is a 24 x 28-in. timber. When the clamshell bucket is in service it is operated by the spud engines.



Engine Operating Brake for Dipper

On account of the geographical location of the harbor the water has a temperature as high as 78 deg., and the condenser required is of extra-large size, having a surface of about 3000 sq. ft. It is 5 ft. in diameter and has 1856 ¾-in. tubes 8 ft. 4 in. long. The boat is equipped with a Davidson vertical beam pump. The boat with two twelve-hour shifts of fifteen men each handles about 6000 yd. of material in each shift.

The hull was built by the Maryland Steel Company for the Michael J. Dady Engineering & Contracting Company, of Brooklyn, N. Y., which equipped it, and is the contractor now operating it for the Cuban Government.

CLEAN RIVER GRAVEL, ⅛ to ¼ in. in diameter, will be permitted for the top coat of concrete paving work now under way in Detroit because the contractors are finding it practically impossible to obtain the granite chips originally specified. The chips are imported from as far away as Massachusetts on the East and Wisconsin on the West. The use of gravel will allow a large quantity of work to be completed this fall. A reduction of 10 cents per yard is to be made from the contract price.

Book Reviews

LABORATORY MANUAL OF TESTING MATERIALS. By William Kendrick Hatt, Professor of Civil Engineering, Purdue University, and H. H. Scofield, Assistant Professor in Laboratory for Testing Materials, Purdue University. Cloth, 7¾ x 5¼ in.; 135 pages; 28 illustrations. New York, McGraw-Hill Book Company. $1.25 net.

(Reviewed by Frank H. Constant, Professor of Structural Engineering, University of Minnesota.)

This little volume is an expansion of an earlier manual published by the authors which was used in several universities. It is the outcome of many years' experience in the laboratory for testing materials of Purdue University. As its name indicates, it is to serve as a laboratory manual to relieve the instructor from the necessity of explaining the details of mechanical procedure, thus giving him opportunity to emphasize the basic principles.

A manual of this sort meets two sources of opposition. One is the feeling among instructors that the student should work out his own salvation pretty largely and that a printed manual is apt to give too many detailed directions. The other is the individual preference of instructors for a certain set of tests which may not agree with that given in the manual. It may be said in answer to the first that some directions are absolutely necessary and that the printed manual is certainly the most convenient form in which to collect such data. The authors have shown the wisdom of long teaching experience in placing before the student, in the briefest form, only such a statement of the object of the test, the material and apparatus to be used, the method of procedure, and the form of computations, diagrams and final report as will give him simply a clear idea of what he is to do. He is not relieved of the necessity of overcoming, by his own initiative, the natural obstacles in manipulation, and in discovering the real purpose of the experiment.

Instructors may not agree upon the proper scope of tests for a laboratory course, but the authors have made a happy selection in covering the usual tests in tension, compression, binding, torsion and impact upon wood, cast iron, steel and cement given in all laboratories. The experiments upon the aggregates used in concrete, leading to scientific proportioning by sieve analyses, are very timely and form a most valuable feature of the work. The list of experiments is somewhat longer than can be covered in the usual laboratory course, allowing the instructor some flexibility in selection and substitution.

The book is of convenient pocket size and contains 135 pages. The first few chapters are general in scope and give general laboratory definitions of stress, elasticity, resilience and other quantities of theoretical import; characteristics of various forms of fracture; and a brief description of various kinds of testing machines, extensometers and other deformation instruments. Chapter 7, devoted to instructions for performing experiments, is subdivided into eight articles of six to eight experiments each, covering experiments upon testing machines, iron and steel, wood, cement, study of aggregates, proportioning of mortars and concretes, tests of concrete and other brittle materials, and tests of road materials. The four appendices cover the common formulæ of mechanics likely to be needed in laboratory practice (given in both the notation of Church and Merriman); strength specifications for steel and iron, and the standard specifications of the American Society for Testing Materials for cement; forms of test pieces, and strength tables for most of the materials of engineering.

While there are several manuals on cement testing and larger treatises on testing of materials, this little volume fills a unique and important place as a simple laboratory manual in the general field of materials testing.

Natural Rock Asphalt and Bitumens. By Arthur Danby, Chemist and Public Consultant on Rock Asphalt and Bitumen. Cloth, 5 x 7½ in.; 244 pages; 15 illustrations. New York, D. Van Nostrand Company. $2.50 net.

(Reviewed by James W. Howard, Consulting Engineer, New York.)

While this book of 244 pages is too small to cover fully the large asphalt industries employing millions of capital and thousands of men in many countries, it includes much of general importance and gives valuable details of a few branches. The book gives information of the sources of supply, preparation and uses especially of bituminous limestone, which the author constantly refers to as rock asphalt and rock asphalt mastics.

There is relatively little on the subject of asphalt or mineral pitch. The natural bituminous limestone, a form of rock asphalt found in many quarries in Europe, has been extensively used for street pavements in Europe for more than sixty years. By the addition of a suitable bitumen in the form of a fluxed asphalt or asphalt cement the rock asphalt has been made into asphalt mastic and used for sidewalks and damp-proof courses for more than a century. Since there are few deposits of bituminous limestone in America, and those located in Utah, Texas and Oklahoma or at points far from large cities, very little of this kind of pavement on streets has been used in America. Bituminous limestone asphalt mastic, with its excellent qualities, has been imported to America and is constantly used for special purposes as in Europe.

The book makes some erroneous statements regarding American deposits of natural bituminous sandstone, which, with few exceptions, makes poorer pavements than bituminous limestone, and which has been used in very small amounts compared with the American system of making sheet asphalt pavements, composed of sand, limestone dust or Portland cement and asphalt cement, and found in several hundred cities to the extent of several thousand miles of streets. Of this there is very little mention in the book and no details are given.

Compiled largely from French and German works, the book supplies in England much information on rock asphalt, its products and uses, and in this respect is valuable there. The book has little value in the United States and Canada, where other forms of asphalt and bituminous roads, pavements, footwalks and waterproofing are extensively used. The nomenclature is not in accord with universal American practice nor in accord with much of the practice of engineers, officials, producers and users of asphalt and its compounds, both natural and artificial, on the Continent of Europe. The reader must constantly remember that the author by "bitumen" means what Americans call refined asphalt and asphalt cement; also by "asphalt" what Americans call rock asphalt—i. e., limestone or sandstone naturally impregnated with asphalt. The book follows the influence of a certain French company which between 1855 and 1866 and at intervals since has issued a great deal of printed matter trying to influence the public to call bituminous limestone "asphalte."

The author mentions a few deposits of "bitumen"—meaning asphalt—in the Western Hemisphere; but evidently did not have Eldridge's large work on asphalt and bituminous deposits of the United States and other independent authorities. He describes crude Trinidad asphalt as "one-third of bitumen, one-third of impurities, decayed wood, vegetable matter, sand and earth, and one-third of water"; and "purified Trinidad asphalt" as "composed roughly of one-half of bitumen and one-half of impurities." He speaks of Bermudez asphalt as "a dangerous rival of Trinidad material," not knowing that the two are controlled by one interest and have been for many years. He briefly mentions the successful use in Great Britain of Cuban, Mexican and California asphalts. The chapter on macadam gives details of English tar macadam, but only outlines, as beginning to be used there, that which is the best practice in America with asphalt-bound roads — constructed by the pouring or penetration and the mixing processes with their mineral aggregates. He says these are still in their infancy in the United Kingdom. The closing chapter gives a valuable outline of the uses of bituminous products in roofing and waterproofing felts and papers.

The book should be had by all interested in natural-rock asphalt mastics; but otherwise it has little value, especially in America.

Water: Its Purification and Use in the Industries. By William Wallace Christie, Consulting Engineer. Cloth, 5½ x 8 in.; 219 pages; 79 illustrations. New York, D. Van Nostrand Company. $2 net.

(Reviewed by Weston E. Fuller, Consulting Engineer, New York)

This book deals largely with the purification of water for industrial purposes. Some parts of it, however, are devoted to purification of water for domestic purposes and to other general water-supply questions.

Many of the conclusions drawn are not in accord with the best practice among waterworks engineers. A number of quotations from other books and papers are given, and in some cases these are misleading. For instance, much of the cost data given for purification works is either out of date or is presented in such a way as to give an erroneous idea.

About half of the book is devoted to water softening, describing various methods in use and going into some detail as to the apparatus involved. One chapter is devoted to definitions of various terms used in water analysis and of the impurities contained in water. Three chapters deal with filtration, ozone treatment, sterilization and other subjects relating to the general question of water supply. Other chapters take up the matter of oil filters, boiler water and the measurement of water. Portions of the book devoted to industrial water supplies, such as water softening, treatment of boilers, etc., are more complete than the remainder.

The book gives evidence of carelessness in compiling. Many subjects are taken up which are treated in a superficial manner.

The Minimum Wage and Syndicalism. By Hon. James Boyle, Formerly Private Secretary of Governor McKinley and American Consul at Liverpool, England. Cloth, 5 x 7¼ in.; 136 pages. Cincinnati, Stewart & Kidd Company. $1 net.

Since its first American demonstration at Lawrence, Mass., there has been much inquiry into the aims and methods of "syndicalism," the generic name for the movement of which the Industrial Workers of the World are the American embodiment. Many articles have been written about it, but few are so likely to impress upon the right-thinking, patriotic American—whether employer or employee—the terrible dangers of the movement as Mr. Boyle's book. His long study of socialism and his intimate acquaintance with British conditions, in which have grown up so many movements affecting labor, well qualify him for discussing the subject. The part of the book devoted to syndicalism covers only 50 pages, but covers it sufficiently for all but the deep student of labor movements. No true citizen can afford to be ignorant of this new manifestation of unrest, and Mr. Boyle's book adequately supplies the needed information.

The section on the minimum wage occupies 86 wages. It states the question briefly, tells the experiences of New Zealand, Australia and England with the minimum wage, discusses the Massachusetts law and draws what conclusions the author feels are justified by the experiences cited. Throughout the presentation is clear and terse, while the statements are supported by copious citations from authorities.

The Theory and Design of Structures. Third Edition. By Ewart S. Andrews. Cloth, 5½ x 8½ in.; 618 pages; 289 figures. London, Chapman & Hall, Ltd. $3.50 net.

(Reviewed by Frank H. Constant, Professor of Structural Engineering, University of Minnesota.)

This is the third edition of this work, which first appeared in 1908. About half of the book is devoted to the principles of stress and strain, centroids, moments of inertia, and the whole subject of beams and flexure. The second half considers framed structures and includes chapters on columns, suspension bridges and arches, the design of steelwork for buildings, roofs, bridges and girders, and also a brief treatment of the stability of measuring structures, such as dams, walls, retaining walls, chimneys, buttresses and masonry arches. About thirty-five pages cover the principles of reinforced-concrete design.

The book is quite monumental in scope and volume, including more than 600 pages of fairly fine text, with numerous illustrations and plates and undoubtedly ranks as a standard English work in the structural field. Its size and scope preclude any attempt at a detailed review, if, indeed, the book is not too well known to require such mention. It is essentially a theoretical treatise, on the whole well written, and each subject is completely developed mathematically. Wherever possible graphical methods are presented. As might be expected from the small amount of space devoted to an important subject, the treatment of reinforced concrete, while sound as far as it goes, omits some essential phases, such as the consideration of shear, diagonal tension, and the design of stirrups and hooped columns. One misses the convenient tables and diagrams always to be found in recent American books on this subject.

The more distinctly practical part of the book on actual design, found in the closing chapters, must be considered entirely inadequate, and one realizes more and more that a text-book on design must have ample space. The plate-girder receives brief treatment, and a numerical example, the design of a 48-ft. span, in which the flanges are made up of 16-in. cover plates and 3½ x 3½ x ½-in. angles, shows very markedly the difference between English and American practice. Plates and cuts of English riveted truss bridges and a brief description of the general method of framing make the American reader wish that the author had gone into the analysis of riveted joints from the European point of view, especially since our own practice is gradually swinging that way.

The additional matter for the present edition is placed in the appendix, and covers Stanton's experiments on wind pressure and the effect upon the stresses of the suction pressure upon the leeward side of a roof truss. There is also a discussion of stresses in curved beams, and a simple parabolic template method of drawing moment diagrams for all classes of uniformly loaded straight beams.

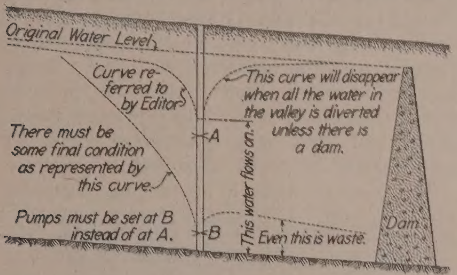
Many numerical examples illustrate and clarify the theory. The book closes with about 120 exercises intended to test the reader's mastery of the subject, chapter by chapter, and his ability to apply the principles to practical problems.

The book is intended for students reading for degree examinations in engineering, as well as for engineers and draftsmen engaged in constructional work.

Letters to the Editor

Irrigation of Santa Cruz Valley

Sir: The note at the foot of my letter to the Editor, published in the Engineering Record of Oct. 25, page 476, regarding "Irrigation of Santa Cruz Valley" shows decidedly that the idea which I wanted to express was entirely lost. There is not only the draw-down curve referred to as being explained on page 201 of the Aug. 23 issue, but there is a second draw-down curve or readjustment of the water levels above and below the line of pumping plants, which is of vital importance to the setting of the pumps and the design of the system. The accompanying sketch will explain more fully the point to which I desired to call attention.


Original Water Level
Curve referred to by Editor
This curve will disappear when all the water in the valley is diverted unless there is a dam.
There must be some final condition as represented by this curve.
A
This water flows on.
Pumps must be set at B instead of at A.
B
Even this is waste.
Dam

Arthur L. Collins,
San Francisco. Consulting Engineer.

Method of Checking Economical Height of Office Buildings

Sir: Some time after reading in your issue of July 5 an abstract of Mr. C. T. Coley's method of checking the economical height of office buildings I was writing my friend, Mr. Benjamin J. Lubschez of A. Van Brunt & Company, architects, Kansas City, Mo., and president of the Kansas City chapter of the American Institute of Architects.

Having collected practically everything which has appeared in the newspapers and magazines in this country during the last twenty years pertaining to the subject of regulating the height of buildings, having made a special study of this subject, and knowing of Mr. Lubschez's interest in the same matter, I asked him what he thought of Mr. Coley's analysis. His reply, it occurs to me, might be of interest to you as a contribution to this important discussion. I therefore quote it practically in full:

"In the article on the economical height of office buildings by Mr. C. T. Coley I find some of the premises radically wrong. Using Mr. Coley's method and what I think are the correct figures where I think his are wrong, the conclusions correspond very nearly with mine, although I arrived at mine in an entirely different way. If you recollect, I said the economical heights were as follows: In Kansas City, 14 to 16 stories; in Chicago, 18 to 20 stories; in New York, 22 to 24 stories. Inasmuch as Mr. Coley's figures are based on New York conditions, we shall check my New York limit, 22 to 24 stories, by his method.

"Mr. Coley's assumed cost of 80 cents to $1 per cubic foot is too high. My investigations show that the very best of office buildings are built for about 50 cents per cubic foot, when cubage includes basements. Inasmuch as Mr. Coley's figures are obviously for cubage above grade only and the cost of basements is absorbed, we should add about 10 per cent. to this, making it 55 cents per cubic foot for cubage above grade. It is assumed in this case that the revenue of basements is also absorbed in the average rate of rental.

"His assumption of 12 ft. for the average height from floor to floor is also too much; 11 ft. are ample as office buildings are now being built. In my judgment, his figures for rent and maintenance are fair and conservative. On account of his building cost being excessive, however, his allowance for depreciation and amortization is too large. I should place it at 20 instead of 30 cents.

"We shall now proceed to use my figures with his method of reasoning. With an average height of 11 ft. to the story, we shall need 11 cu. ft. of building to produce 1 sq. ft. of floor area. Then 11 ÷ 0.65 (rentable area being 65 per cent of total) = 16.9 cu. ft. of building needed to produce 1 sq. ft. of rentable area. At 55 cents per cubic foot this would cost $9.295. Interest on this at 5 per cent is 46.5 cents per year. This plus 20 cents depreciation and amortization makes 66.5 cents, which, added to Mr. Coley's maintenance charge of $1.50, makes the total cost per square foot of rentable area $2.165. Subtracting this from $3.50 leaves a net profit per year of $1.335 per square foot per story of rentable area. The total expense to overcome being $33.23, the number of stories is $33.23 \div 1.335 = 24.9$, or, say, 25 stories. By taking slightly lower basic figures, such as 10 ft. for average story heights and 50 cents for the cost per cubic foot, the result would be from 22 to 23 stories, which is not far from my original estimate.

"After all, the empirical way in which Mr. Coley has treated the problem is not fair nor at all accurate. The conditions of the problem vary markedly with different sizes and shapes of sites, these affecting the planning and consequent rentable areas. The only way to arrive at a fair solution is to figure several projects from scale-plan diagrams, making careful cube estimates of cost, careful estimates of rentable areas and probable rents from them and so on, and then average the results for the different projects.

"My figures for several proposed office-building schemes, so estimated, show that not only are the heights arrived at the enocomical ones, but that they are also the heights of maximum income. Buildings much higher than these show a lesser rate of income, while according to Mr. Coley's formula, the higher a building is above his economical height, the greater will be the rate of income. My conclusions are easily accounted for by the following reasons: Above a certain height the higher a building is, (1) the less will be the ratio of rentable space, especially in the lower stories, where rent is the highest, because much greater space will be needed for elevator accommodations and lobbies and increased sizes of piers and columns, and this loss of space in the most valuable part of the building will make a very marked difference in the rate of income; (2) the more will be the proportionate cost of foundations, and (3) the greater will be the proportionate cost of maintenance.

"A few actual figures from one of the schemes I spoke of may be of some interest to you. The building in question is 87 x 115 ft. The ground rental three years ago, when the figures were made, was $35,000 a year. For a 6-story building, the net income would be 4.88 per cent on the total cost; for a 12-story building, 11.3 per cent; for a 16-story building, 11.5 per cent; for a 24-story building, 8.6 per cent, and for a 30-story building, 7.5 per cent.

"These figures would vary somewhat now, as the ground value has increased. They fairly represent, however, the average in the business district of Kansas City. They might vary two stories one way or the other."

D. Knickerbacker Boyd,
Philadelphia. Architect.

Extra Top Width for New Railroad Embankments

Sir: In your issue of Oct. 25, page 459, is a discussion of shrinkage of new railroad embankments and extra widths and heights to allow for shrinkage. This is one of the hard things engineers must encounter in building railroad embankments or any other fills. To make proper allowance for shrinkage one must have at his command a large experience in this line.

There are many factors entering into the problem. The material may be sand, clay, loam, rock or something else. The fill may be made by wheelbarrows, drag scrapers, wheel scrapers or wagons, or dumped from cars on temporary trestles. Each and every one of these conditions must be met by different coefficients for shrinkage. Furthermore, this shrinkage allowance is not simply for height, but must be for volume. A fill shrinks on its sides as well as on top. The only formula necessary is to know how to sidestake and calculate volume by old and well-established methods.

It may be premised that settlement, like shrinkage of wood, never ceases. A fill built of rock not liable to decompose, or of sand, may be expected to remain as it is built, no matter how it is placed. If rock is liable to disintegrate, shrinkage will occur. I have found that ordinary material, such as clay, loam or mixtures, will generally measure about 5 per cent in embankments over the borrow pits or excavations from which it was taken, if placed by carts, drag-scrapers or wheelers. If deposited by wheelbarrows it may be expected to shrink about 50 per cent. If placed by cars from a trestle an allowance of 10 per cent should be made for shrinkage. In section staking for a fill, the stakes should be set for the grade height of the embankment plus the allowance for shrinkage. This gives a full-width roadbed with shrinkage, and when settlement takes place a full-width roadbed is secured.

Now as to the propriety of an allowance for shrinkage, to which one of the engineers discussing the matter objects—it is all important to do so. That the surplus material will be cut off for a fast-approaching track is no potent reason why it should not have been put on. The track sometimes follows construction rapidly, but very often months elapse before it is laid. The great desideratum is a full-width road-bed. Unless a shrinkage allowance is provided for this is never obtained. Those familiar with track-laying know that it is almost impossible to prevent the sides of the fill from being robbed to get dirt for replacing the settlement. Now when extra material has been put on and cut down, there is never a lack of material for the surfacing of the track. Frequently it is not entirely cut off, and the track, if closely following, is not surfaced until later on, when it is necessary to cut off whatever excess height may be left.

Thomas B. Lee,
Chief Engineer, Piedmont & Northern Lines.
Charlotte, N. C.

[While it is true that a fill tends to shrink equally in all directions, it cannot do so, because the length must remain constant. Furthermore, a new fill will slip and slide, in addition to shrinking, until the ultimate slopes are quite different from the original slopes. Thus the final top width will be much less than would be the case if the embankment simply shrunk uniformly. This sliding action is the foundation for the formulæ derived by Professor Fish.—Editor.]

Location of Neutral Axis in Curved or Bent Beams

Sir: While doing some work on truck equalizing bars the error in the usual definition for "neutral axis" was impressed upon me. Text books define the neutral axis for beams under bending as the line passing through its center of gravity. As a matter of fact this definition is true only when the plane through the center of gravity is at right angles to the plane of action of the loading. Whenever the beam deflects, or when it is of such construction (see figure) that there is a component of the load acting along its center of gravity, then the neutral axis will be shifted from its otherwise normal position. This same error is frequently made in current periodicals, and a beam, like the one shown, will have its center of gravity, *C. G.*, line labeled *N. A.*, neutral axis, throughout its entire length.

In the figure is represented the conventional design of one type of truck equalizing bars used mainly on passenger cars and locomotive tenders. The location of the neutral axis or plane of zero stress can be easily computed. Using the notation of the figure where S_{tb} equals tensile stress from bending; S_{tr}, resultant tensile stress; S_{cb}, compressive stress from bending; S_{cr}, resultant compressive stress; S_t, direct tensile stress; *N. A.*, neutral axis; *C. G.*, center of gravity; *W*, weight of load; *L*, distance of *W* from support; *I*, moment of inertia of section; *a*, area of section; e_c, distance from *C. G.* to fiber in compression; and e_t, distance from *C. G.* to fiber in tension, the following formulæ are obtained:

$$S_{cr} = Me_c/I - W \sin\alpha/a \qquad \text{(A)}$$

$$S_{tr} = Me_t/I + W \sin\alpha/a \qquad \text{(B)}$$

At the point of zero stress both equations (A) and (B) become zero, so that

$$Me_c/I - W \sin\alpha/a = 0$$

Letting e_0 represent the distance from the center of gravity of the section to the point of zero stress, and substituting $W x$ for M, we have $W x\,(e_c/I) = W \sin\alpha/a$, or (compressive stress from bending) = (direct tensile stress), from which

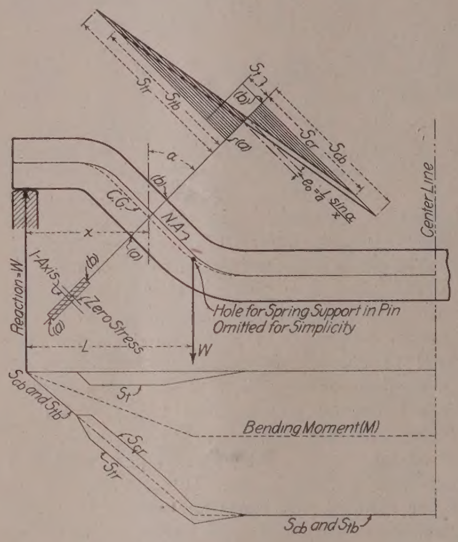
$$e_c = e_0 = \frac{I}{a}\,\frac{\sin\alpha}{x} \qquad \text{(C)}$$

Similarly, from equation (B) the following expression can be derived:

$$e_t = e_0 = -\frac{I}{a}\,\frac{\sin\alpha}{x} \qquad \text{(D)}$$

The negative value obtained from equation (B) merely signifies that the distance is measured from the center of gravity toward the side in compression.

It is to be noted that in computing the bending stresses the moment of inertia is taken about the center of gravity, since the stresses due to bending will still be in proportion to the distance of the fiber from the center of gravity of the section. Along that part of the beam subjected to a direct stress, it might be said that the *C. G.* is the "apparent neutral axis," while the line of zero stress is the "resultant neutral axis." But whatever distinction is made, it seems to me that, for the sake of precision and accuracy, the usual definition for neutral axis should be modified, since it is obvious that one line cannot satisfy the conditions that it pass through the centers of gravity and the points of zero stress at the same time, when these two sets of points do not coincide. The term "neutral axis" signifies fundamentally the "line of zero stress." Its definition, then, should be such that it will apply to all beams, irrespective of their shape or character of loading.



Offset of Neutral Axis in Beams

Moline, Ill. J. Paul Shamberger.

[We are of the opinion that the term "neutral axis" means a plane (warped or flat) in which are located all fibers of the beam subjected to neither compression nor tension, and not a line passing through the center of gravity.—Editor.]

Sherman Law and Agreements between Engineers Not to Enter Competitions

Sir: In a communication in the Engineering Record of Sept. 20, page 335, from Mr. Luten appears the following statement: "Mr. Drane was apparently employed by the commission as their consulting engineer; that is, as their technical adviser to help them secure the most efficient bridge. Instead of accepting that capacity he insists upon acting as their designing engineer, a position properly filled by a specialist in bridge design. A commission of laymen properly advised by a competent consulting engineer need have no hesitancy in securing real competition by including competition in design along with labor and materials."

That sounds very well. The writer during the past year was approached by two boards of elected officials with a view to retaining him as their engineer for the design and construction of reinforced-concrete bridges. He advised them that a number of qualified specialists were connected with companies having a great deal of experience in such work and that the best thing they could do would be to have their engineer act merely as a consulting engineer. His duties would consist in the preparation of proper specifications and checking of such designs as might be offered. He informed them that it would not be necessary to check each design, but merely the design of the lowest bidder. If this design were found weak, or inconsistent with the specifications, it should be rejected and the designs of the next lowest bidders checked until a design was found which complied in all respects with the specifications. If no design was found which came within the limits of cost it then would be time to prepare a special design and invite bids from every contractor who thought he could erect such a structure.

The cost of such a proceeding compared with the cost of making a design was fully gone into. Then, after a proper design was obtained, the consulting engineer could be retained to supervise construction. The result was that other engineers were employed. They prepared designs and bids were called for. Companies making a specialty of such work submitted designs of their own with bids and obtained the work. The commissioners were out the extra fee charged, which, of course, the public paid for. Not only that, but the specialists employed by the bridge companies were able to prove, even to laymen, that there was grave question as to the real ability of the engineers employed to prepare the plans for open competition. The bridges are being erected under the supervision of these engineers, but the commissioners have lost confidence in them and other outside engineers have been employed to check their superintendents.

Mr. Luten has the right idea as to the proper function of the consulting engineer, but the majority of elected officials are as yet not educated up to the real value of a consulting engineer and his function in such matters. The effect upon the writer has been such that hereafter he will accept work when it comes his way and prepare plans so that general contractors may compete with specialists in design and construction. It is hardly right to ask the public to foot the bills for this unnecessary work, but if the servants elected by the people insist upon it, the consulting engineer must either accept the employment in the way it is offered him or try to make a living in some other way. The average elected official does not want an engineer to give him any advice until he has been actually retained, and to give in advance the advice the writer attempted to give is always resented. It is even considered by these men as a tacit admission that the engineer offering it feels himself not competent to do the work, otherwise he would not voluntarily throw up the chance to make a big fee and instead accept a fee for merely looking over plans and acting as a special adviser. For about 95 per cent of the bridge work done in this country special designs should not be prepared. This is shown by the fact that the Federal Government and a number of States issue standard plans and standard specifications.

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